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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XLI—No. 1

January, 1915

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XLI—No. 1

JANUARY, 1915

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1915

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TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS: Nelson P. Lewis, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, John A. Bensel.

TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, Frank M. Kerr.

ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

ON FLOODS AND FLOOD PREVENTION: C. McD. Townsend, John A. Bensel, T. G. Dabney, C. E. Grunsky, Frank M. Kerr, Morris Knowles, J. B. Lippincott, Daniel W. Mead, John A. Ockerson, Arthur T. Safford, Charles Saville, F. L. Sellew.

TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Robert W. Hunt, George W. Kittredge, Paul M. LaBach, C. G. E. Larsson, William McNab, G. J. Ray, F. E. Turneaure, J. E. Willoughby.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
CABLE ADDRESS....."Ceas, New York."

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

December 16th, 1914.—The meeting was called to order at 8.30 P. M.; Director George W. Fuller in the chair; Chas. Warren Hunt, Secretary; and present, also, 162 members and 26 guests.

A paper by D. D. Clarke, M. Am. Soc. C. E., entitled "Submerged Pipe Work at Portland, Oregon", was presented by the Secretary, who also read communications on the subject from Messrs. Clemens Herschel, Ben S. Morrow, and F. M. Randlett. The paper was discussed by Messrs. T. H. Wiggin, W. E. Spear, W. W. Brush, J. P. Hogan, C. D. Ward, Kenneth Allen, and R. C. Kellogg.

A paper by C. E. Smith, M. Am. Soc. C. E., entitled "History of Little Rock Junction Railway Bridge of the St. Louis, Iron Mountain and Southern Railway Company, Over the Arkansas River, at Little Rock, Arkansas, 1883-1914," was presented by the Secretary, who

also presented a communication on the subject from Mr. C. D. Purdon. The paper was discussed orally by Messrs. Henry H. Quimby and T. Kennard Thomson.

The Secretary announced the following deaths:

William Roberts Eckart, of Palo Alto, Cal., elected Member, January 5th, 1881; died December 8th, 1914.

Henry Martyn Rood, of Port Chester, N. Y., elected Member, December 3d, 1890; died November 24th, 1914.

William Paul Hufschmidt, of Wiggins, Colo., elected Associate Member, October 4th, 1910; died August 13th, 1914.

Wallace King, Jr., of New York City, elected Associate, December 4th, 1900; died October 31st, 1914.

Adjourned.

January 6th, 1915.—The meeting was called to order at 8.30 p. m.; Vice-President Gardner S. Williams in the chair; Chas. Warren Hunt, Secretary; and present, also, 140 members and 25 guests.

The minutes of the meetings of November 18th and December 2d, 1914, were approved as printed in *Proceedings* for December, 1914.

A paper by Samuel Tobias Wagner, M. Am. Soc. C. E., entitled "The Water-Proofing of Solid Steel-Floor Bridges" was presented by the author and illustrated with lantern slides.

The Secretary presented communications on the subject from Messrs. Albert J. Himes, William S. Babcock, Jonathan Jones, J. Lee Allen, and Glenn B. Woodruff.

The paper was discussed orally by Messrs. A. W. Carpenter, G. J. Ray, J. H. Fichthorn, J. B. W. Gardiner, F. Auryansen, T. Kennard Thomson, E. W. DeKnight, Henry W. Hodge, Henry H. Quimby, and the author.

The Secretary made the following announcement:

"A noteworthy incident in the history of the Profession of Engineering in the United States will be the inauguration of The Engineering Foundation on January 27th, 1915, in the Auditorium of the United Engineering Society in New York City.

"The Engineering Foundation is the name given to a fund to be administered for the advancement of the Arts and Sciences connected with Engineering and the benefit of Mankind, the basis of which is the initial gift of a considerable sum by a noted engineer.

"The American Society of Civil Engineers, The American Institute of Mining Engineers, The American Society of Mechanical Engineers, and The American Institute of Electrical Engineers, are to be represented equally in the administrative Board of The Engineering Foundation by election by the Board of Trustees of The United Engineering Society which has been made the custodian of the fund. All members

and friends of the Engineering Profession are invited to these inaugural ceremonies."

On account of lack of time, the Secretary was unable to announce the elections and transfers by the Board of Direction at its meeting of January 6th, 1915, but they are here included for the purpose of the record.

Elected as Members

Edmund Raymond Kinsey, St. Louis, Mo.
Wilfrid Van Nest Powelson, New York City
Frank Reeves, Buenos Aires, Argentine Republic
Clarence Mord Rogers, Daytona, Fla.
William Edward Rolfe, St. Louis, Mo.
Sturgis Hooper Thorndike, Boston, Mass.

Elected as Associate Members

Elbert Christopher Aldrich, Auburn, N. Y.
George Fredson Baine, Buenos Aires, Argentine Republic
Alfred Fortin Beck, Norfolk, Va.
William Hatrick Becker, Groveland, Cal.
Elbert Calvin Brown, Manila, Philippine Islands
Frank Jerome Carew, New York City
Johnson Chapman, Barfield, Ark.
Norman Moore Chivers, Kings Bluff, N. C.
Richard Vincent Collins, Hornell, N. Y.
Edward Clarence Dohm, Olympia, Wash.
Charles Raymond Ege, Olympia, Wash.
John Arthur Elliott, Wyeth, Ore.
Walter Wier Johnston, Anniston, Ala.
Karl Raymond Kennison, Providence, R. I.
Alfred Fellows Masury, New York City
Arthur Onderdonk, New York City
Bernard Arnold Ruge, New York City
Charles Boone Sadler, Baltimore, Md.
Leroy Clarke Smith, Lansing, Mich.
George Gardner Underhill, Santa Rosalia, Chih., Mexico
Anders Bagge Villadsen, Salt Lake City, Utah
Gerrit Waalkes, Mt. Vernon, Ohio
John Palmer Walker, Juneau, Alaska
Carl August Wigholm, Oakland, Cal.
Clarence Richard Young, Toronto, Ont., Canada

Elected as Associates

John Alexander Britton, San Francisco, Cal.
Clark Edwin Mickey, Lincoln, Nebr.

Elected as Juniors

John Henning Anderson, Chicago, Ill.
Wilbur Earl Bickerton, Boston, Mass.
Walter Louis Connelly, Saginaw, Mich.
William Bullard Durant, Turners Falls, Mass.
Harold Laselle Fiske, Schenectady, N. Y.
Manuel Font, San Juan, Porto Rico
Walter Cleary Hopkins, Baltimore, Md.
Edwin Ambler Ingham, Berkeley, Cal.
Frederick Martin Kipp, Jr., Baltimore, Md.
John Lowery, Jr., Ambridge, Pa.
George Alfred Maney, Minneapolis, Minn.
Albertis Montgomery, Minneapolis, Minn.
Melvin Lyman Moone, Lansing, Mich.
Robert Bruce Murdock, Watertown, N. Y.
Jesse Thomas Osborn, Chicago, Ill.
Edmund Quincy, New York City
Richard Bennett Smith, Culebra, Canal Zone, Panama
James Leland Stanford, New York City
John Wright Taussig, New York City
Francis Montgomery Veatch, Lawrence, Kans.

Transferred from Associate Member to Member

Dudley Tibbits Corning, Troy, N. Y.
Louis Adolph Mitchell, Anderson, Ind.
Philip à Morley Parker, London, England
Stuart Sims, Iowa City, Iowa
Carl Henry Stengel, New York City
Albert Edwin Waldron, Washington, D. C.

Transferred from Junior to Associate Member

Arthur Frederick Blight, Los Angeles, Cal.
John Dubuis, Salem, Ore.
Walter Hanna Dunlap, Roanoke, Va.
Charles Lacey Hall, Manila, Philippine Islands
Frank Sampson Mason Harris, Oakland, Cal.
George William Howson, San Francisco, Cal.
Elmer Alfred Porter, Salt Lake City, Utah
Ernest Walker Sawyer, Newcastle, N. B., Canada
Arthur Louis Sherman, Patchogue, N. Y.
Nathan Newton Tiffany, East Hampton, N. Y.
Hugh Lemuel Wiley, Portland, Ore.

The Secretary announced the following deaths:

ROBERT MAXSON GREENE, of Winnipeg, Man., Canada, elected Associate Member, June 1st, 1898; Member, September 1st, 1908; died December 5th, 1914.

ELIPHALET W. BLATCHFORD, of Chicago, Ill., elected Fellow, February 6th, 1873; died January 25th, 1914.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

January 6th, 1915.—The Board met at 3 P. M.; Vice-President Smith in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bates, Bensel, Bush, Cooley, Cummings, Edwards, Endicott, Fuller, Haskell, Hodge, Leonard, Montfort, Swain, Thomson, Tuttle, and Williams.

*Regulations for Administering Admission Requirements were adopted.

†Rules for the Appointment, Government, and Procedure of Special Committees were adopted.

The Chairman was authorized to appoint Tellers to canvass the Ballot for Officers, and these Tellers were authorized to begin the canvass of the Ballots on the evening of Tuesday, January 19th, 1915.

A Report from the Committee to Recommend the Award of Medals and Prizes for 1914 was received, and the following awards were made in accordance with the recommendations of the Committee:

The Norman Medal to Caleb Mills Saville, M. Am. Soc. C. E., for his paper entitled "Hydrology of the Panama Canal."

The J. James R. Croes Medal to J. B. Lippincott, M. Am. Soc. C. E., for his paper entitled "Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct."

The Thomas Fitch Rowland Prize to H. T. Cory, M. Am. Soc. C. E., for his paper entitled "Irrigation and River Control in the Colorado River Delta."

The James Laurie Prize to Samuel Tobias Wagner, M. Am. Soc. C. E., for his paper entitled "The Elevation of the Tracks of the Philadelphia, Germantown and Norristown Railroad, Philadelphia, Pa."

The Collingwood Prize for Juniors to J. S. Longwell, Jun. Am. Soc. C. E., for his participation in the preparation of the paper (prepared jointly by him and Mr. W. G. Steward who is not a member of the Society), entitled "Experiments on Weir Discharge."

The Constitution of the recently formed Cleveland Association of Members of the Society was approved.

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* These will be printed in the February Number of *Proceedings*.

† See page 7.

The invitation to this Society to participate in the administration of the fund to be known as the Engineering Foundation, tendered to this Society by the United Engineering Society, was accepted.

$500 was appropriated for the use of the Joint Committee of the five National Societies on the reception of Foreign Engineers.

The Board approved the Ballot for Officers as issued.

Action was taken in regard to members in arrears for dues.

The resignations of 10 Members, 12 Associate Members, 2 Associates, and 14 Juniors, were accepted.

Ballots for membership were canvassed, resulting in the election of 6 Members, 25 Associate Members, 2 Associates, and 20 Juniors, and the transfer of 11 Juniors to the grade of Associate Member.

Six Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

February 3d, 1915.—8.30 P. M.—At this meeting, two papers will will be presented for discussion, as follows: "Nomographic Solutions for Formulas of Various Types," by R. C. Strachan, M. Am. Soc. C. E.; and "Rivers and Railroads in the United States," by William W. Harts, M. Am. Soc. C. E.

Mr. Strachan's paper was printed in *Proceedings* for December, 1914, and Mr. Hart's paper is printed in this number of *Proceedings*.

February 17th, 1915.—8.30 P. M.—At this meeting, a paper by William G. Grove, Esq., and Henry Taylor, Assoc. M. Am. Soc. C. E., entitled "Reconstruction of the Norfolk and Western Railway Company's Bridge Over the Ohio River at Kenova, West Virginia," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

RULES ADOPTED BY THE BOARD OF DIRECTION FOR THE APPOINTMENT, GOVERNMENT, AND PROCEDURE OF SPECIAL COMMITTEES, JANUARY 6TH, 1915.

Authorization: The authorization of Special Committees is prescribed by Article VI, Section 12, of the Constitution of the Society. All Special Committees are subject to these regulations. The object of a Special Committee is to investigate and report upon the engineering subjects defined in its title or in an explanatory statement by the Board.

Personnel: Appointments on Committees are made by the Board of Direction from the Membership of the Society, and will continue from year to year, except when changes are announced by the Board. Before Members are appointed the Board shall have their assurance that the work of the Committee will have their interest and co-operation. A member of any Committee who does not take an active interest in its work and does not regularly attend its meetings during the year, nor render adequate service by correspondence, may be relieved, and the vacancy filled by the Board. A member of a Special Committee may retire as such upon request given in writing to the Secretary.

After the Board of Direction has appointed the Chairman of a Special Committee, the Board will consult with the Chairman as to the personnel of the Committee. Additional appointments on existing Committees shall be made only on the recommendation of, or with the approval of, such Committees.

Funds: The Chairman of each Special Committee as soon after the appointment of the Committee as possible shall prepare a budget of itemized expenses covering the requirements of the Committee for the remainder of that year; thereafter he shall submit a budget not more than 10 days after the Annual Meeting of the Society. These budgets shall be scrutinized and such appropriations approved as may, in the judgment of the Board of Direction, be considered expedient. All bills submitted by a Special Committee shall be approved by its Chairman and Secretary.

Expenses: Expenses for items other than stationery and postage shall not be assumed by the Society, unless such expenditures were incurred in pursuance of previous authorization of the Board or referred to in the annual budget.

Extraordinary expenses, such as purchase of instruments, salaries of special employees and traveling expenses must be specifically authorized and approved by the Chairman of the Committee concerned. The Chairman of each Special Committee shall return its tangible property and records to the Society on the completion of its work.

Officers: The officers of Special Committees shall be a Chairman, appointed by the Board of Direction, and a Secretary, elected by the Committee, who shall assume their duties immediately after appointment.

Duties of Officers: The Chairman shall preside at all meetings of the Committee, appoint all Sub-Committees unless otherwise directed, and be *ex officio* a member of all Sub-Committees.

The Secretary will be expected to attend all meetings of the Committee, keep a roll call of the Members in attendance, and record the minutes in a book kept for that purpose.

(*a*) He shall conduct the correspondence, and receive all communications addressed to the Committee, and furnish each Member a complete copy of all communications.

(*b*) He shall read the minutes at all meetings, and shall issue notices for all meetings and promptly inform Sub-Committees of their appointments and duties.

(*c*) He shall keep a memorandum of all expenses of the Committee. He shall prepare an abstract of the minutes of all meetings and forward same to the Board of Direction for publication in the monthly *Proceedings*, and shall perform such other duties as may be required of him.

Meetings: The Committee shall hold a meeting as soon as practicable after its appointment, at which time Sub-Committees on Organization and Scope of the work contemplated shall be appointed.

At least two regular meetings of the Committee shall be held each year, the time and place to be fixed by the Chairman. Special Meetings may be called at the option of the Chairman or at the request of the majority of the Committee; due notice of such meetings to be given to the Secretary of the Society and to the members of the Committee.

Reports: No final action on Committee reports may be taken except at a meeting called for that purpose. Final reports shall receive the approval of a majority of those voting. Dissenting Members may present minority reports individually or jointly. Those refraining from voting shall be named in the report.

The Reports of Special Committees shall be filed with the Secretary of the Society at least 60 days before the Annual Meeting.

Amendments: Changes in the above Rules may be adopted by any Committee only upon the approval of the Board of Direction.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the

second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

December 12th, 1914.—The meeting was called to order; President Vincent in the chair; Roger W. Toll, Secretary; and present also 18 members and guests.

The minutes of the meeting of November 21st, 1914, were read and approved.

It was announced that, until further notice, the weekly luncheons would be held on Wednesdays at 12.30 P. M., at the Albany Hotel.

H. S. Crocker was appointed to represent the Association at the meeting of the Presidents of the Local Associations to be held in New York City on January 20th-21st, 1915.

A paper by W. W. Curtis, on "Woods and Wood Preservation," was presented by the author, who illustrated his remarks with lantern slides. The subject was generally discussed by those present.

A vote of thanks was extended to Mr. Curtis for his paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on December 29th, 1913, the new Chairman, John Ruddle, was installed, and Messrs. Park A. Dallis and G. R. Solomon were appointed members of the Executive Committee. T. P. Branch was elected Secretary.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

At its meeting of January 6th, 1915, the Board of Direction considered and approved the proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

The Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.) has been organized with the following officers: Frank M. Kerr, President; J. F. Coleman and W. B. Gregory, Vice-Presidents; A. M. N. Blamphin, Treasurer; and L. C. Datz, Secretary.

Northwestern Association

At its meeting of November 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.). F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The Philadelphia Association of Members of the American Society of Civil Engineers was organized on December 22d, 1913. The meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized, with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

St. Louis Association

At its meeting of October 7th, 1914, the Board of Direction considered and approved the proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year: Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

(Abstract of Minutes of Meetings)

November 30th, 1914.—The meeting was called to order at the College Club, at 12.15 P. M.; President Ernest B. Hussey in the chair; Carl H. Reeves, Secretary; and present, also, 33 members and guests.

The minutes of the meeting of October 26th, 1914, were read and approved.

The resignation of Mr. S. B. Phillips, as a member of the Association, was accepted.

Discussion of the Engineers' License Bill was opened by Messrs. Joseph Jacobs and Robert Howes, presenting the affirmative and negative sides, respectively, and the subject was discussed further by Messrs. Ralston, Blackwell, Fuller, Reeves, and others.

In accordance with the action of the Association at its meeting of October 26th, 1914, inviting the members of various local technical societies to attend this meeting and discuss the feasibility of the federation of such societies, representatives were present from the Pacific Northwest Society of Engineers, the Engineers' Club, the Seattle Section of the American Institute of Electrical Engineers, the Seattle Section of the American Institute of Mining Engineers, the American Chemical Society, and the Washington Association of Engineers. Discussion of the subject was opened by Mr. S. C. Lindsay, of the American Institute of Electrical Engineers, who was followed by Messrs. Ralston, Magnussen, Blackwell, Dimock, and others.

On motion, duly seconded, it was decided to request each of the local technical societies to appoint three members as a preliminary committee to formulate plans for federation by which each organization would retain its identity, the first meeting of such committee to be subject to the call of the President of the Association.

Mr. Howes called attention to an article in *The Sun* of October 28th, 1914, in which it was stated that University professors would investigate and report on small water-power possibilities in the State free of charge, except for traveling and incidental expenses. Professor Fuller explained that it was not the intention of the University to usurp the field of the engineer in private practice, that the project applied only to small water-powers, which ordinarily would not be used, and that if developments were made, it was the intention of the University to suggest that the owner call in an engineer.

Mr. Ralston called attention to the fact that the Spokane Association was considering the proposed Water Code, and inasmuch as the 1915 Legislature would soon convene, it was felt that the subject should be taken up and reviewed carefully.

The President appointed Professor Fuller a member of the Conference Committee to fill the vacancy caused by the resignation of Mr. E. J. McCaustland.

Adjourned.

December 28th, 1914.—The meeting was called to order at 12.15 P. M., at the College Club; Ernest B. Hussey, President, in the chair; Carl H. Reeves, Secretary; and present, also, 20 members and guests.

The minutes of the meeting of November 30th, 1914, were read and approved.

The President read letters from the Secretary of the Society relating to the scope and functions of Local Associations and announcing a meeting of the Presidents of the Associations to be held in New York City, on January 19th, 1915.

On motion, the Conference Committee was asked to formulate matters to be presented by the representative of the Association at that meeting.

On motion, duly seconded, the President was authorized to appoint a representative to such meeting, in case he was unable to attend.

On motion, duly seconded, the President appointed a Committee of five to make arrangements for the Annual Meeting of the Association, as follows: Messrs. Samuel H. Hedges, Chairman, A. W. Münster, A. S. Downey, Joseph Jacobs, and R. H. Ober.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 515 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers. Ulysses B. Hough is President.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Valuation of Public Utilities

December 28th, 29th, and 30th, 1914.—The eighth meeting of the Committee was held in Boston. Present, F. P. Stearns (Chairman), Leonard Metcalf, Charles S. Churchill, William G. Raymond, Henry E. Riggs, Jonathan P. Snow, and William J. Wilgus (the full membership of the Committee).

The draft of two chapters of the report, discussed at the previous meeting, was again discussed and revised. The outline of the first half of the report, which had been previously prepared and discussed

in writing, was considered, and the various topics therein were fully discussed. Chapters of the report were assigned to different members for the preparation of drafts to serve as a basis for written discussion in the interval between meetings, and for oral discussion at the next meeting.

Special Committee on Steel Columns and Struts

November 19th, 1914.—The meeting was held at the Bureau of Standards, Washington, D. C. Present, A. L. Bowman (Chairman), James H. Edwards, Rudolph P. Miller, George H. Pegram, and Lewis D. Rights. The following visitors were present: Mr. F. Auryansen, Bridge Engineer, Long Island Railroad; Mr. W. H. Moore, Chairman of the Sub-Committee on Columns for the American Railway Engineering Association; and Dr. G. R. Olshausen, Physicist of the Bureau of Standards.

The Committee inspected a column, Type 10, slenderness ratio, $85\ \frac{L}{R}$, which was being placed in the testing machine.

The Secretary read a letter from the Secretary of the Society reporting the request of the Board of Direction that the minutes of the meetings of Special Committees be forwarded for publication in *Proceedings*.

The Committee discussed the Progress Report. The Secretary read a letter from Charles F. Loweth, suggesting that the Board of Direction be asked to change the title of the Committee by omitting from it the words "and Struts." It was decided not to request the change in title.

The Progress Report was approved.

With reference to the suggestion (in the minutes of the meeting of October 6th) as to the advisability of dropping back the loads to 1 000 lb. per sq. in. from 5 000, 10 000, and 15 000 lb. per sq. in., it was decided to leave the matter in the hands of Dr. Olshausen, who will make some investigations and communicate with the Secretary.

The Committee then witnessed the breaking of the test column, and adjourned to hold an informal meeting on the train from Washington to New York.

At this adjourned meeting, Mr. Miller submitted some diagrams of a proposed method of plotting the 8-in. strain gauge measurements. This method was discussed, and Mr. Miller was requested to elaborate upon it, and submit the results to the next meeting of the Committee.

Special Committee on Floods and Flood Prevention

December 15th, 1914.—The meeting was held in the office of the Chairman, 428 Custom House, St. Louis, Mo. Present, C. McD. Townsend (Chairman), John A. Ockerson, and Morris Knowles.

The following communications were read and discussed:

1.—Report of the sub-committee on Rivers Emptying into the Gulf of Mexico, presented by C. McD. Townsend, and a discussion of that report by T. G. Dabney.

2.—Report of the sub-committee on the Ohio River and Its Tributaries, presented by John A. Ockerson.

3.—Two papers from the sub-committee on Rivers Emptying into the Pacific Ocean, viz., (*a*) Flood Control on the Colorado River, by F. L. Sellew, and some remarks thereon by C. E. Grunsky; (*b*) Flood Control of the Sacramento and San Joaquin Rivers, by C. E. Grunsky.

4.—A letter from Daniel W. Mead on the flood conditions in China.

The Committee learned indirectly that J. B. Lippincott had prepared a paper on Rivers Emptying into the Pacific Ocean, but the paper had not been received.

After an informal discussion of these papers and allied subjects, lasting throughout the day, it was decided to prepare a general report in accordance with the views presented, which will then be submitted to the other members of the Committee for their consideration.

Special Committee on Materials for Road Construction

January 21st, 1914.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), A. W. Dean, Nelson P. Lewis, George W. Tillson, and A. H. Blanchard.

The Chairman announced that the Board of Direction had added to the Committee three members, George W. Tillson, Nelson P. Lewis, and Charles J. Tilden, and presented for consideration his resignation as chairman. It was decided that the present chairman and secretary be continued.

The Committee discussed the scope of the work and the formation of sub-committees.

January 22d, 1914.—The meeting was held at the office of Mr. Nelson P. Lewis, New York City. Present, W. W. Crosby (Chairman), A. W. Dean, Nelson P. Lewis, George W. Tillson, and A. H. Blanchard.

The following sub-committees were appointed:

New Forms.—A. W. Dean, Nelson P. Lewis, and H. K. Bishop.

Tests and Analyses.—George W. Tillson, A. H. Blanchard, and W. W. Crosby.

Cost Forms.—Nelson P. Lewis, Charles J. Tilden, and A. W. Dean.

April 6th, 1914.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, Nelson P. Lewis, Charles J. Tilden, and A. H. Blanchard.

Progress reports were submitted on Cost Forms and on Tests and Analyses. A report on New Forms, was presented by letter.

The report of the sub-committee on New Forms, as amended, was adopted.

The Secretary was authorized to re-draft a circular letter to be sent out with the new forms, and also authorized to incur the expense of printing 2 000 circular letters and 10 000 of the new forms.

The scope of the work of the sub-committee on Cost Forms was discussed.

June 4th, 1914.—The meeting was held at the Belvedere Hotel, Baltimore, Md. Present, W. W. Crosby (Chairman), H. K. Bishop, Charles J. Tilden, and A. H. Blanchard.

The routine work of the Committee, and the Chairman's outline of the Committee Report to be presented to the Society in January, 1915, were discussed.

September 19th, 1914.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard.

It was decided that the present sub-committees would be continued, with the understanding that the subjects referred to them would be considered under the new title of the Committee, "Special Committee on Materials for Road Construction." (The original title was Special Committee on Bituminous Materials for Road Construction.)

Progress reports of the sub-committees on Cost Forms and Tests of Bituminous Materials were presented and accepted.

The Secretary was instructed to request the Board of Direction to authorize a special meeting of the Society, to be held on the Friday following the Annual Meeting of 1915, for the discussion of the report of the Committee, and to provide opportunity for subsequent written discussion.

The Secretary was instructed to submit to the Board of Direction the request that 3 000 copies of the report, in addition to the number necessary to supply the members of the Society, be printed for the use of the Committee in connection with its work in securing data relative to the use of materials for road construction, and also in connection with the special meeting in January, 1915, if the latter is authorized.

The tentative report was discussed, paragraph by paragraph, amended, or modified, with a view of preparing a re-draft for discussion at a subsequent meeting.

October 17th, 1914.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard.

A communication from the Secretary of the Society authorizing the holding of a special meeting of the Society on Friday, January 22d, 1915, for the discussion of the Committee's report, was read.

A report from the sub-committee on Tests of Bituminous Materials was presented, amended, and tentatively accepted.

The draft of the "Progress Report on Materials Other than Bituminous" was presented, amended, and tentatively accepted.

November 2d, 1914.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard.

The draft of the section of the report dealing with Nomenclature was presented, amended, and tentatively accepted, except in the case of a few definitions which were not considered.

The section of the report relating to Tests of Bituminous Materials was reconsidered, amended, and tentatively accepted.

The Secretary was instructed to revise the section of the report dealing with Analyses and Tests of Bituminous Materials.

November 14th, 1914.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard.

The Secretary reported that arrangements had been made to print 1 000 copies of the 1915 report of the Committee and to hold the type so that additional copies could be printed if required.

The draft of the report, including Appendix C, relating to Analyses and Tests of Bituminous Materials, was discussed, amended, and adopted.

It was decided to omit the section of the report dealing with Non-Bituminous Road Materials.

A vote of thanks to Mr. Crosby, Chairman of the Committee, as an appreciation of the extraordinary amount of work which he has done on the 1915 report, was carried unanimously.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont , Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31st, 1914.

In compliance with the Constitution, the Board of Direction presents its report for the year ending December 31st, 1914.

MEMBERSHIP

The changes in membership are shown in the following table:

	Jan. 1st, 1914.			Jan. 1st, 1915.			Losses.				Additions.		Totals.		
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.	Increase.
Honorary Members.	.....	5	5	.....	6	6	....	..	..	..	1	.....	....	1	1
Corresponding " .	.....	1	1	.....	1	1	....	..	..	..	.....	.....	....	....	....
Members.....	648	2 627	3 275	658	2 744	3 402	1	14	11	42	†96	*99	68	195	127
Associate Members.	547	2 437	2 984	569	2 679	3 248	96	26	32	16	§108	326	170	434	264
Associates.....	77	97	174	69	97	166	4	5	4	3	.....	8	16	8	‡8
Juniors.....	156	660	816	160	680	840	104	21	29	2	.....	180	156	180	24
Fellows.....	5	12	17	5	10	15	....	..	..	2	.....	.....	2	....	‡2
Totals.....	1 433	5 839	7 272	1 461	6 217	7 678	205	66	76	65	205	613	412	818	406

* 1 Reinstatement.
† 96 Associate Members.
‡ Decrease.
§ 4 Associates, 104 Juniors.

It will be noted that the net increase in membership for the year is 406.

The number of applications received during 1914 was 1 000: 730 for admission and 270 for transfer.

The losses by death reported during the year number 65, and are as follows:

Members (42): Ricardo Manuel Arango, Howard Elmer Arthur, Benjamin Thomas Buffinton, Edward Cook Burns, Robert Cooke Clarkson, Joseph P. Cotton, William Crawford, Horace Crosby, William Roberts Eckart, James Joseph Ferris, Charles Francis, James Lewis Frazier, Martin Luther Gardner, Webster Gazlay, Emil Gerber, Justus Herbert Grant, Bernard Richardson Green, Robert Maxson Greene, John Willis Hays, William August Hunicke, Hurd Clarence Hurd, William Dunbar Jenkins, Thomas H. Johnson, Emil Kuichling, Frank Parsons Lant, Dickinson MacAllister, David Neilson Melvin, Benjamin Franklin Morse, George Alfred Nelson, Alfred Noble, Henry Lewis Oestreich, Peter Alexander Peterson, Arthur Louis Phillips, Frederic John Henry Rickon, Wallace Berkley Riegner, Henry Martyn

Rood, Edmund Job Steere, Lucian Arnold Taylor, Fred Thompson, Joseph Tintorer y Giberga, Nathaniel Turner, George Westinghouse.

Associate Members (16): Ralph Albree, Lawrence Calvin Brink, Duncan Hugh Campbell, Horace Arthur Cook, George John Couchot, Philip Chapin Davis, Murray Forbes, Leon Lincoln Gay, Claude Sanford Haynes, Roger Tifft Holloway, William Paul Hufschmidt, Jesse Sidwell Matson, Edgar Henry Mix, John Marvin Peters, Philip Morris Pritchard, William de Hertburne Washington.

Associates (3): Wallace King, Jr., William Churchill Oastler, John McGaw Woodbury.

Juniors (2): Warren Austin Gates, Aaron J. Zabriskie.

Fellows (2): William Richardson Belknap, Eliphalet W. Blatchford.

LIBRARY

The total contents of the Library and the increase during the year, are shown in the following statement:

	Total Contents.	Increase during 1914.
Bound volumes	24 632	1 275
Unbound volumes	46 255	1 850
Specifications	7 688	268
Maps, photographs, and drawings	4 889	215
Total	83 464	3 608

Of these, 1 482 were donations received in answer to special requests; 69 were donations from publishers; 1 761 were donations received in regular course, and 296 were purchased.

The value of accessions to the Library during the year is as follows, each accession having been valued separately as received:

Donations and exchanges (estimated value)	$2 680.77
296 volumes purchased (cost)	898.67
Binding 429 volumes	502.82
Total	$4 082.26

The following amounts have been expended upon the Library during the year:

Purchases, subscriptions, and binding	$1 401.49
Fixtures, supplies, express charges, etc.	107.66
Total	$1 509.15

The card index now contains about 92 000 cards.

During the year 84 new bibliographies (containing 4 670 separate references) have been made, copies of 21 searches made in previous

years have been furnished, 8 of these having been brought up to date. The total cost of this work, $813.51, has been charged to those for whom it was undertaken.

The total attendance in the Reading Room and Library during the year, was 5 001. This does not include those who use the Library during the semi-monthly meetings.

The Board believes that there are many members of the Society who fail to take advantage of the benefits of its Library facilities because they are not aware of just what those facilities are.

In the belief that our large technical reference Library is one of the most important of the assets of the Society, the Library Committee during the year has looked into the question of its condition with a view of undertaking any improvement which might make it more useful to Engineers. Many people who use this and other technical libraries frequently have been interviewed, and their criticism, as well as suggestions for improvements, have been sought. Other libraries of similar general character have been visited and their work examined in detail. One suggestion that the so-called Dewey System of indexing would be more comprehensive and easy to use has received careful consideration and investigation, with the result that the Committee is satisfied that the system now in use is more simple, more comprehensive, and generally more satisfactory.

One improvement that has been started is the preparation and filing of index cards of all references to current technical literature which are published monthly, and it is believed that this will soon become a valuable labor-saving device, as, if it is kept up, it will be an easily consulted reference to more than a hundred of the best of the engineering publications.

SPECIAL COMMITTEES

During the year the Board has taken up the question of extending the work of the Society through its Special Committees, of which we now have ten, and to this end has authorized, by making up of budgets, the payment of mileage (on the same basis as mileage is now paid to members of the Board of Direction) for attendance at Committee meetings. Payments have also been made for necessary material for conducting experiments and employing suitable assistants for carrying them out. The whole subject of methods under which the work of Special Committees to report on Engineering subjects should be carried out is still under consideration by the Board, and it is hoped that this may soon be formulated, and that this branch of the Society's work may be much stimulated and result in great benefit to the Profession. There has certainly been much more activity in the work of the Special Committees this year than ever before.

PUBLICATIONS

During the year, ten numbers of *Proceedings,* one volume of *Transactions,* and one List of Members, have been issued.

In *Proceedings* the list of references to current engineering literature has been continued, and has covered 148 pages and contained 6 624 classified references to 108 periodicals.

The stock of the various publications of the Society, kept on hand for the convenience of members and others, now amounts to 170 390 copies, the cost of which to the Society, for paper and press work only, has been $26 781.13.

During the year 5 129 volumes of *Transactions* have been bound for members and others in standard half-morocco and cloth bindings.

SUMMARY OF PUBLICATIONS FOR 1914.

	Issues.	Average Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volume LXXVII)..	1	8 200	1 968	40	356
Proceedings (Monthly numbers)..	10	8 150	4 076	41	626
Constitution and List of Members..	1	8 400	318	..	1
Total.....	12	.....	6 362	81	983

The cost of publications has been:

For Paper, Printing, etc., *Transactions* and *Proceedings*....	$35 270.11
For Plates and Cuts.....	2 963.32
For Boxes, Mailing Lists, Copyright, and Sundry Expenses..	850.46
For 25 900 Extra Copies of Papers and Memoirs.....	1 495.87
For List of Members.....	2 213.38
Total.....	$42 793.14
Deduct amount received from sale of publications.....	4 453.85
Net expenditure for publications for 1914.....	$38 339.29

From the Annual Report for 1913, the following statement covering the publications for that year is taken:

SUMMARY OF PUBLICATIONS FOR 1913.

	Issues.	Average Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volume LXXVI)...	1	7 700	2 302	76	655
Proceedings (Monthly numbers)...	10	7 625	3 184	104	497
Constitution and List of Members..	1	7 850	296	..	1
Total.....	12	.....	5 782	180	1 153

The cost of publications has been:

For Paper, Printing, etc., *Transactions* and *Proceedings*...	$38 698.05
For Plates and Cuts	3 964.16
For Boxes, Mailing Lists, Copyright, and Sundry Expenses..	667.56
For 41 150 Extra Copies of Papers and Memoirs	2 106.31
For List of Members	1 964.84
Total	$47 400.92
Deduct amount received from sale of publications	4 908.46
Net expenditure for publications for 1913	$42 492.46

Although a comparison of the respective cost of publications for these two years appears to show a large saving in 1914, it should be stated that it is not possible to make an exact comparison between any two sets of publications. One item alone, that of Plates and Cuts, shows a greater cost in 1913 of $1 000, and to this must be added the cost of printing the extra Plates, as well as the cost of insertion, folding, etc.

MEETINGS

Twenty-seven meetings were held during the year, as follows: At the Annual Meeting, 1; at the Annual Convention, 3; and 23 other meetings, all of which were held at the Society House.

At these meetings there were presented 29 formal papers, 9 of which were illustrated with lantern slides. There were also 5 papers published which were not presented for discussion at any meeting of the Society. The number of members and others who took part in the preparation or discussion of these papers was 275.

The Forty-sixth Annual Convention was held at Baltimore, Md.

The total attendance at the 27 meetings held was about 5 450. The registered attendance at the Annual Meeting was more than 1 400, and at the Annual Convention, 346, but there were many guests present at all these meetings, and also members who failed to register.

At each of the ordinary semi-monthly meetings held during the year collations have been served, and these have been paid for out of the Society funds, in accordance with the action of the Annual Meeting of 1912.

MEDALS AND PRIZES

For the year ending with the month of July, 1913, prizes were awarded as follows:

The Norman Medal to J. V. Davies, M. Am. Soc. C. E., for his paper entitled "Air Resistances to Trains in Tube Tunnels."

The J. James R. Croes Medal to B. F. Cresson, Jr., M. Am. Soc. C. E., for his paper entitled "The Problem of the Lower West Side Manhattan Water-Front of the Port of New York."

The Thomas Fitch Rowland Prize to Burgis G. Coy, Assoc. M. Am. Soc. C. E., for his paper entitled "The Laramie-Poudre Tunnel."

The James Laurie Prize to M. M. O'Shaughnessy, M. Am. Soc. C. E., for his paper entitled "Construction of the Morena Rock Fill Dam, San Diego County, California."

FINANCES

The mortgage debt of the Society has been reduced $20 000 during the year, from $80 000 to $60 000, and the Reserve Fund increased by the purchase of ten New York City Bonds (par value, $10 000) so that this fund now consists of ninety New York City Bonds (par value, $90 000), on which the interest received is $1 350 more than that paid on the mortgage. This statement shows a Reserve Fund of $30 000 in excess of the mortgage, which Reserve Fund should be steadily added to in order to offset the depreciation on the Society House.

The attention of members is invited to the Secretary's statement of receipts and disbursements, and to the general balance sheet which accompanies it, in which the financial condition of the Society is shown.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction.

CHAS. WARREN HUNT,
Secretary.

JANUARY 6TH, 1915.

GENERAL BALANCE SHEET, DECEMBER 31st, 1914.

ACCOMPANYING THE REPORT OF THE SECRETARY.

ASSETS.		
Three Lots (Actual cost, $189 632.11) (Estimated value)		$375 000.00
Society Building (cost)		170 955.59
Furniture (cost)		20 792.96
Publications on hand (inventoried cost)		26 781.13
New York City non-taxable bonds (cost)		90 392.50
Library: Cash expended for books, etc.	$20 028.33	
Donations (estimated)	68 114.46	88 142.79
Due from Members		9 899.18
Due from Non-Members		592.27
Interest accrued on investment		1 049.98
Cash		17 253.83
		$800 860.23

LIABILITIES.	
Dues for 1915 paid in advance	$28 576.74
Mortgage debt	60 000.00
Interest accrued on mortgage	200.00
Funds invested in Society House, lots and Library*	28 015.78
Herbert Stewart Library Fund	1 997.50
Gen. Joseph G. Swift Library Fund	998.75
Surplus (including Reserve Fund of $87 396.25)	681 071.46
	$800 860.23

We have examined the books and accounts of the American Society of Civil Engineers, for the year ended December 31, 1914, and certify that the foregoing Balance Sheet is in accordance therewith, and, in our opinion, correctly states the condition of the Society's affairs, as shown by the books.

79 WALL STREET, NEW YORK,
JANUARY 6TH, 1915.

MARWICK, MITCHELL, PEAT & CO.,
Chartered Accountants.

* Compounding Dues Fund, $11 755.00; Norman Medal Fund, $1 000.00; Rowland Prize Fund, $1 222.50; Collingwood Prize Fund, $1 000.00; Fellowship Fund, $13 038.28.

REPORT OF THE SECRETARY FOR THE

To the Board of Direction of the

Gentlemen:—I have the honor to present a statement of Receipts and Disbursements for the fiscal year of the Society, ending December 31st, 1914. I also append a general balance sheet showing the condition of the affairs of the Society.

Respectfully submitted,
Chas. Warren Hunt,
Secretary.

Receipts.

Balance on hand December 31st, 1913, in Bank, Trust Company, and in hands of Secretary		$18 433.43
Entrance Fees	$15 050.00	
Current Dues	81 536.75	
Past Dues	3 820.88	
Advance Dues	28 576.74	
Certificates of Membership	656.80	
Badges	3 037.50	
Sales of Publications	4 453.85	
Library	820.86	
Annual Meeting	2 269.50	
Binding	4 820.33	
Interest	4 229.76	
Compounding Dues	825.00	
Miscellaneous	363.73	
		$150 461.70

$168 895.13

YEAR ENDING DECEMBER 31st, 1914.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

DISBURSEMENTS.

Salaries of Officers	$15 100.00	
Mileage of Directors	2 903.26	
Mileage of Nominating Committee	482.70	
Work of Committees (Including Mileage, $1 009.72)	3 599.91	
Clerical Help	23 113.48	
Caretaking	1 738.30	
Publications	42 793.14	
Postage	6 879.67	
General Printing	2 755.18	
Office Supplies	1 276.33	
House Supplies	70.47	
Library	1 401.49	
Library Maintenance	107.66	
Badges	2 023.89	
Certificates of Membership	445.00	
Binding	3 620.12	
Prizes	239.61	
Annual Convention	426.85	
Annual Meeting	3 550.12	
Maintenance of House	260.91	
Heat, Light and Water	873.56	
Furniture	129.36	
Interest	2 774.44	
Insurance	161.00	
Current Business	3 081.65	
Petty Expenses	239.81	
International Engineering Congress, 1915	405.60	
Alfred Noble Memorial	953.67	
Advanced on Members' Accounts	47.29	
Bond and Mortgage	20 000.00	
Reserve Fund	9 967.50	
Miscellaneous	219.33	
		$151 641.30
Balance on hand December 31st, 1914:		
In Union Trust Company	$10 412.32	
In Garfield National Bank	5 341.51	
In hands of Secretary	1 500.00	
		17 253.83
		$168 895.13

REPORT OF THE TREASURER.

In compliance with the provisions of the Constitution, I have the honor to present the following report for the year ending December 31st, 1914:

Balance on hand December 31st, 1913			$18 433.43
Receipts from current sources, January 1st to December 31st, 1914			150 461.70
Payment of Audited Vouchers for Current Business, January 1st to December 31st, 1914		$121 673.80	
Payment on principal of bond and mortgage.		20 000.00	
Purchase of bonds, Reserve Fund		9 967.50	
Balance on hand December 31st, 1914:			
In Union Trust Company	$10 412.32		
In Garfield National Bank	5 341.51		
In hands of the Secretary	1 500.00	17 253.83	
		$168 895.13	$168 895.13

Respectfully submitted,

JOHN F. WALLACE,
Treasurer.

NEW YORK, JANUARY 6TH, 1915.

ACCESSIONS TO THE LIBRARY

(From December 2d, 1914, to January 2d, 1915)

DONATIONS*

AMERICAN SEWERAGE PRACTICE:

Volume I, Design of Sewers. By Leonard Metcalf and Harrison P. Eddy, Members, Am. Soc. C. E. Cloth, 9½ x 6½ in., illus., 10 + 747 pp. New York and London, McGraw-Hill Book Company, Inc., 1914. $5.00.

This book, it is stated, is the first of three volumes to be issued by the authors, on the principles of theory and rules of practice in the design and operation of sewerage works. In this first volume, they have discussed the design of sewerage systems in detail, supplementing the subject-matter, much of which, it is stated, has been tested practically in their own work, with data, tables, and diagrams to illustrate standard practice. The second volume, it is said, is to deal with the Construction of Sewers and the third with the Disposal of Sewage. The Chapter headings are: The Lessons Taught by Early Sewerage Works; The General Arrangement of Sewerage Systems; Flow of Water in Pipes and Channels; Velocities and Grades; Measurement of Flowing Water; Quantity of Sewage; Precipitation; Formulas for Estimating Storm-Water Flow; The Rational Method of Estimating Storm-Water Run-Off in Sewer Design; Gaging Storm-Water Flow in Sewers; Sewer Pipe; The Design of Masonry Sewers; Examples of Sewer Sections and the Loads on Sewers; The Analysis of Masonry Arches; Street Inlets, Catch-Basins, and Manholes; Junctions, Siphons, Bridges and Flushing Devices; Regulators, Overflows, Outlets, Tide Gates, and Ventilation; Sewage Pumping Stations; Index.

THE PRESERVATION OF STRUCTURAL TIMBER.

By Howard F. Weiss. Cloth, 9½ x 6¼ in., illus., 18 + 312 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $3.00.

In studying the problems arising during nine years of official work in connection with the wood-preservation industry, the author states that the need of a book on the subject in its broadest sense was repeatedly felt. It was also thought that a textbook on timber preservation would be of assistance to students in forestry and engineering schools where a knowledge of wood utilization is desirable and often necessary. The subject-matter of this book has been taken largely, it is stated, from the author's lecture notes before the civil engineering students at the University of Wisconsin, and his aim has been to present reliable detailed information on the subject which will be of value and use to engineers, foresters, lumbermen, and students. In the Appendices the author has given the minor wood-preserving processes patented, proprietary, and minor wood preservatives used in the United States; lists of manufacturers of creosote and zinc chloride, and lists of wood-preserving plants and fire-proofing plants in the United States; amount of wood preservative used and timber treated, as well as a list of companies equipped to build wood-preserving plants in the United States; specifications for the analysis of creosote; United States Patents in wood preservation; method of analyzing zinc chloride; records of life of timbers, mine timbers, paving blocks, poles, cross-ties, etc. The Contents are: Introduction; Factors Which Cause the Deterioration of Structural Timber; The Effect of the Structure of Wood Upon its Injection with Preservatives; The Preparation of Timber for its Preservative Treatment; Processes Used in Protecting Wood from Decay; Preservatives Used in Protecting Wood from Decay; The Construction and Operation of Wood Preserving Plants; Preserving the Life of Cross-ties from Decay and Abrasion; Prolonging the Life of Poles and Cross Arms from Decay and Insects; Prolonging the Life of Fence Posts from Decay; Prolonging the Life of Piling and Boats from Decay and Marine Borers; Prolonging the Life of Mine Timbers; Prolonging the Life of Paving Blocks; Prolonging the Life of Shingles; Prolonging the Life of Lumber and Logs; The Protection of Timber from Fire; The Protection of Wood from Minor Destructive Agents; The Strength and Electrolysis of Treated Timber; The Use of Substitutes for Treated Timber; Appendices; Index.

Gifts have also been received from the following:

Alabama-R. R. Comm. 1 bound vol.
Am. Inst. of Elec. Engrs. 1 pam.
Am. Inst. of Min. Engrs. 2 vol.
Am. Medical Assoc. 1 pam.
Am. Soc. of Heating and Ventilating Engrs. 1 pam.
Am. Telephone & Telegraph Co. 1 vol., 1 pam.

* Unless otherwise specified, books in this list have been donated by the publishers.

Architekten und Ingenieur-Verein zu Hamburg. 2 bound vol.
Arizona-Agri. Exper. Station. 2 pam.
Arizona-State Board of Equalization. 1 pam.
Arizona-State Engr. 1 vol.
Armour Inst. of Technology. 1 pam.
Assoc. Parisienne des Propriétaires d'Appareils à Vapeur. 1 vol.
Baltimore & Ohio R. R. Co. 2 pam.
Booth, W. H. 1 pam.
Boston, Mass.-Directors of the Port of Boston. 2 pam.
Boykin, J. C. 1 bound vol., 4 pam.
Bradbury, E. G. 1 pam.
Brown, A. G. 2 pam.
Bureau of Ry. Economics. 14 pam.
California-R. R. Comm. 1 pam.
California-State Water Comm. 1 pam.
California, Univ. of. 1 vol.
Canada-Conservation Comm. 1 bound vol.
Canada-Dept. of Marine and Fisheries. 1 bound vol.
Canada-Dept. of Mines. 1 bound vol., 1 vol., 1 pam.
Canada-Dept. of the Interior. 1 pam.
Canada-Geol. Survey. 1 pam.
Carnegie Endowment for International Peace. 1 pam.
Chicago, Ill.-Municipal Reference Library. 1 pam.
Clarke, D. D. 1 bound vol.
Colorado-Public Utilities Comm. 5 pam.
Colorado-State Highway Comm. 1 pam.
Colorado School of Mines. 2 pam.
Connecticut-Public Utilities Comm. 1 vol., 1 map.
Cooper Union for the Advancement of Science and Art. 1 vol.
Detroit, Mich.-Board of Water Commrs. 1 pam.
Dist. of Columbia-Engr. Commr. 1 vol.
Dodge, Grenville M. 3 bound vol., 1 pam.
Dooling, Peter J. 1 pam.
Donham, Grenville M. 1 bound vol.
Eidlitz, Marc, & Son. 1 bound vol.
Engrs. Club of Baltimore. 1 bound vol.
Engrs. Club of Dayton. 1 pam.
Everett, Mass.-City Clerk. 1 bound vol.
Fletcher, Duncan U. 2 vol.
Flinn, Alfred D. 1 pam.
Freeman, John R. 1 bound vol.
Gardner, Augustus P. 1 pam.
Holliday, Alex. R. 1 pam.
Illinois-Bureau of Labor Statistics. 2 bound vol.
Illinois-Coal Min. Investigations. 1 pam.
Illinois-Public Utilities Comm. 1 bound vol., 2 pam.
Illinois-Rivers and Lakes Comm. 1 bound vol.
Illinois-Secy. of State. 1 bound vol.
Illinois-State Water Survey. 1 bound vol.
Indiana-Dept. of Geology and Natural Resources. 1 bound vol.
Inst. of Industrial Research. 1 pam.
Inst. of Min. Engrs. 1 pam.
Jacksonville, Fla.-Trustees for the Water-Works and Improvement Bonds. 1 vol.
Kansas-Dept. of Labor and Industry. 1 bound vol.
Kansas-Public Utilities Comm. 4 pam.
Kansas State School of Mines and Metallurgy. 1 pam.
Kentucky-Geol. Survey. 1 pam., 1 map.
Kershaw, G. Bertram. 2 pam.
Koninklijk Instituut van Ingenieurs. 1 pam.
Lehigh & Hudson River Ry. Co. 1 pam.
Livingston, Robert E. 1 pam.
Louisiana-Conservation Comm. 1 pam.
Louisiana-State Board of Appraisers. 2 pam.
Luedecke, Carl. 2 pam.
Lunberg, A. P., & Sons. 1 pam.
McCumber, P. J. 1 pam.
Maine-Executive Dept. 1 pam.
Marstrand, O. J. 2 pam.
Massachusetts-Chf. of the Dist. Police. 1 bound vol., 10 pam.
Massachusetts-State Forester. 1 pam.
Massachusetts-State Highway Comm. 2 pam.
Mass. Inst. Tech. 1 vol.
Mexican Ry. Co., Ltd. 2 pam.
Missouri-Agri. Exper. Station. 1 pam.
Missouri, Univ. of. 1 vol.
Montana-State Dept. of Public Health. 1 pam.
Montgomery, Ala.-City Auditor. 1 pam.
Moore, William S. 7 vol.
National Assoc. of Port Authorities. 2 pam.
National Elec. Light Assoc. 4 bound vol.
National Fire Protection Assoc. 1 pam.
Natural Ice Assoc. of America. 1 bound vol.
Nevada-Public Service Comm. 4 pam.
New Hampshire-Forestry Comm. 1 pam.
New Hampshire-Public Service Comm. 2 pam.
New Jersey Sanitary Assoc. 1 pam.
New London, Conn.-Water and Sewer Dept. 1 pam.
New Mexico-State Corporation Comm. 1 bound vol.
New York City-Dept. of Docks and Ferries. 1 bound vol.
New York City-Public Works Commr. 6 bound vol., 3 vol.
New York State-Conservation Comm. 6 pam.
New York State-Dept. of Labor. 5 pam.
New York State-Public Service Comm., First Dist. 2 pam.
New York State-Public Service Comm., Second Dist. 2 pam.
New York City Record. 1 bound vol.
New York Historical Soc. 1 pam.
New York State Coll. of Forestry. 6 pam.
New York Univ. 2 pam.
North Carolina-Corporation Comm. 1 bound vol.
North Carolina-Geol. and Economic Survey. 1 vol., 2 pam.
North Dakota-R. R. Commrs. 1 map.
North Dakota State School of Forestry. 1 pam.
Ohio-Auditor of State. 1 vol.
Ohio-Commrs. of Public Printing. 1 map.
Ohio-Industrial Comm. 17 pam.
Ohio-State Fire Marshal. 1 pam.
Ohio-State Library. 7 pam.
Ontario, Canada-Bureau of Mines. 1 vol.
Ontario, Canada-Ry. and Municipal Board. 1 vol.
Oregon-Conservation Comm. 1 pam.
Oregon, Univ. of. 1 pam.
Pasadena, Cal.-Auditor. 2 pam.

Paschke, Theodore. 1 pam.
Pennsylvania-Board of Game Commrs. 1 vol.
Pennsylvania-Bureau of Rys. 1 bound vol.
Pennsylvania-Public Service Comm. 4 pam.
Pennsylvania-Senate Librarian. 1 bound vol.
Pennsylvania-State Highway Dept. 1 pam.
Pennsylvania-Water Supply Comm. 2 bound vol.
Perrine, Harold. 1 pam.
Philadelphia, Pa.-Dept. of City Transit. 1 pam.
Pittsburgh, Pa.-Carnegie Library. 1 pam.
Punjab, India-Irrig. Dept. 1 bound vol.
Purdue Univ. 1 vol.
Rhode Island-Commrs. of Inland Fisheries. 1 bound vol.
Rhode Island-Public Utilities Comm. 1 pam.
Royal Inst. of British Archts. 1 vol.
Royal Soc. of Canada. 4 pam.
Saskatchewan, Canada-Dept. of Public Works. 1 pam.
Smithsonian Institution. 3 pam.
Soc. of Am. Foresters. 1 pam.
South Dakota School of Mines. 1 vol., 2 pam.
Tasmania-Dept. of Mines. 1 pam.
Terre Haute, Ind.-City Clerk. 4 pam.
Texas-Dept. of Agri. 1 pam.
Texas, Univ. of. 3 pam.
Travelling Engrs. Assoc. 1 bound vol.
Union Coll. 1 vol.
U. S.-Bureau of Mines. 1 vol.
U. S.-Bureau of Yards and Docks. 1 pam.
U. S.-Census Bureau. 1 pam.
U. S.-Chf. of Engrs. 3 vol., 15 specif.
U. S.-Coast and Geodetic Survey. 1 bound vol., 2 pam.
U. S.-Commr. of Lighthouses. 1 pam.
U. S.-Dept. of Agri. 2 pam.
U. S.-Director of Public Roads. 1 pam.
U. S.-Forest Service. 1 pam.
U. S.-Geol. Survey. 8 vol., 20 pam., 25 maps.
U. S.-Interstate Commerce Comm. 1 bound vol., 1 pam.
U. S.-Library of Congress. 1 bound vol., 1 pam.
U. S.-Naval Observatory. 1 pam.
U. S.-Reclamation Service. 5 pam.
U. S.-Secy. of Commerce. 1 vol.
U. S.-Secy. of the Interior. 12 pam.
U. S.-Supt. of Documents. 1 pam.
Victoria, Australia-Mines Dept. 1 pam.
Virginia-Corporation Comm. 1 pam.
Wabash R. R. Co. 1 pam.
Ward, C. D. 2 pam.
Washington-State Highway Dept. 1 vol.
Washington Soc. of Engrs. 1 pam.
Washington, Univ. of. 2 pam.
West Ham, England-Town Clerk. 3 bound vol., 2 pam.
West Virginia-Chf. Road Engr. 7 pam.
Western Australia-Geol. Survey. 6 pam.
Wisconsin-Agri. Exper. Station. 1 pam.
Wisconsin-R. R. Comm. 1 pam.
Wisconsin-State Superv. of Inspectors of Illuminating Oils. 1 pam.
Yale Univ. 1 vol.

BY PURCHASE

Hydraulic Motors, with Related Subjects Including Centrifugal Pumps, Pipes, and Open Channels, Designed as a Text-Book for Engineering Schools. By Irving P. Church. New York and London, 1911.

The Incorporated Association of Municipal and County Engineers: International Engineering Congress (Glasgow), 1901. Proceedings of Section 7: Municipal. Edited by Thomas Cole. New York and London, 1901.

The Gas Engine in Principle and Practice, Including Comparison of the Two-Cycle and Four-Cycle Types of Internal Combustion Engines, with Description of Various Designs, etc. By A. H. Goldingham. New York and London, 1912.

Some Considerations Regarding Cast-Iron and Steel Pipes. By John Sharp. New York and London, 1914.

Elements of Modern Field Artillery, U. S. Service. By Harry G. Bishop. Menasha, Wis., 1914.

A Text-Book of Military Hygiene and Sanitation. By Frank R. Keefer. Philadelphia and London, 1914.

American Society for Testing Materials: Proceedings of the Seventeenth Annual Meeting Held at Atlantic City, N. J., June 30th–July 3d, 1914; Vol. 14, Part 1, Committee Reports. Edited by the Secretary. University of Pennsylvania, Philadelphia, Pa., 1914.

Isolation Hospitals. By H. Franklin Parsons. Cambridge, 1914.

The Bacteriological Examination of Food and Water. By William G. Savage. Cambridge, 1914.

The Railways of South and Central America: A Manual Containing Statistics and Other Information Concerning the Important Railways of South and Central America, Mexico, and the West Indies. By Frederic M. Halsey. New York, 1914.

Textbook of Mechanics; Vol. 5: Hydraulics. By Louis A. Martin, Jr. New York and London, 1914.

A Text-Book of General Bacteriology. By Edwin O. Jordan. Fourth Edition, Revised. Philadelphia and London, 1914.

The Blasting of Rocks in Mines, Quarries, Tunnels, etc.: A Scientific and Practical Treatise for the Use of Engineers and Others Engaged in Mining, Quarrying, Tunneling, etc., and for Mining and Engineering Students. By Albert W. Daw and Zacharias W. Daw. Second Edition, with Addenda. New York and London, 1909.

A Short Table of Integrals. By B. O. Peirce. Second Edition, Revised. Boston and New York, 1910.

SUMMARY OF ACCESSIONS

(From December 2d, 1914, to January 2d, 1915)

Donations (including 15 duplicates)	422
By purchase	14
Total	436

MEMBERSHIP

(From December 4th, 1914, to January 7th, 1915)

ADDITIONS

MEMBERS

		Date of Membership.
HAMILTON, CHARLES FRAZINE. Gen. Mgr., The Northwestern Constr. Co., Box 621, Franklin, Pa.		June 24, 1914
KAST, CLARKE NIGHTINGALE. Asst. Field Engr., Interstate Commerce Comm., Wells Fargo Bldg., San Francisco, Cal.	Jun.	June 4, 1901
	Assoc. M.	June 7, 1905
	M.	Dec. 2, 1914
LONGLEY, FRANCIS FIELDING. Cons. Engr. (Hazen, Whipple & Fuller), 30 East 42d St., New York City.	Jun.	Feb. 28, 1905
	Assoc. M.	Mar. 2, 1909
	M.	Dec. 2, 1914
THOMAS, RALEIGH COLSTON. Chf. Engr., Electrical Comm. of Baltimore, City Hall Annex No. 2, Baltimore, Md.		June 24, 1914
THOMAS, RALPH DANFORD. Asst. Engr., St. Anthony Falls Water Power Co. and Minneapolis Mill Co., Minneapolis, Minn.	Assoc. M.	May 6, 1908
	M.	Dec. 2, 1914
WIGGIN, ERNEST WOODBURY. Cons. Engr., 506 Cutler Bldg., New Haven, Conn.	Jun.	May 5, 1896
	Assoc. M.	May 7, 1902
	M.	Dec. 2, 1914
WORTHINGTON, IRVING. Asst. Engr. and Water Supt., Fresno Canal & Irrig. Co., Griffith-McKenzie Bldg., Fresno, Cal.		June 24, 1914

ASSOCIATE MEMBERS

ASHLEY, CARL. Field Engr., The T. A. Gillespie Co., 16 East 37th St., New York City.		Dec. 2, 1914
BECTON, JOHN LELAND. Wilmington, N. C.		Dec. 2, 1914
BILLWILLER, ERNEST OSWALD. Civ. and Hydr. Engr. (Henderson & Billwiller), 242 East Main St., Stockton, Cal.	Jun.	Nov. 8, 1909
	Assoc. M.	Dec. 2, 1914
BROWN, ARCHIBALD ALEXANDER. Asst. Engr., Panama-Pacific International Exposition Co., 6433 Harmon Court, Oakland, Cal.		Sept. 2, 1914
CAMPBELL, WILLIAM COOPER. Supt. of Public Works, and City Engr., Box 507, Columbus, Ga.		Dec. 2, 1914
CLARK, JOHN JAMES. Designing Engr., William B. Ittner, 5744 De Giverville Ave., St. Louis, Mo.	Jun.	Sept. 3, 1913
	Assoc. M.	Dec. 2, 1914
DAY, WARREN ELLIS. Care, Utah Power & Light Co., 503 Kearns Bldg., Salt Lake City, Utah.	Jun.	Dec. 6, 1910
	Assoc. M.	Dec. 2, 1914
DE L'EAU, JOHN PEALE. Civ. and Hydr. Engr.; Engr., Union Sugar Co., Betteravia, Cal.		Dec. 2, 1914
FEELEY, WILLIAM PATRICK. Div. Engr., Great Lakes Dredge & Dock Co., 1100 D. S. Morgan Bldg., Buffalo, N. Y.	Jun.	May 2, 1911
	Assoc. M.	Nov. 4, 1914

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
FERNALD, GORDON HILDRETH. Asst. Engr. and Asst. Supt., Sewage Disposal Comm., City of Fitchburg, 172 South St., Fitchburg, Mass		Dec. 2, 1914
FRASER, DE GENNES. Chf. Engr., Cook County Council; Acting Engr., Gisborne Borough Council, P. O. Box 22, Gisborne, New Zealand		Nov. 4, 1914
GASCOIGNE, GEORGE BRADLEY. Asst. City Engr., Room 417, City Hall, Cleveland, Ohio		Nov. 4, 1914
GELDER, ROYAL WILLIAM. Engr. (Lincoln & Gelder), 1112 Eleventh St., Greeley, Colo		Nov. 4, 1914
GLANDER, JOHN HENRY, JR. Asst. Engr., Charles W. Leavitt, Jr., 1107 Empire Bldg., Birmingham, Ala	Jun. Assoc. M.	May 3, 1910 Nov. 4, 1914
GOSS, OLIVER PERRY MORTON. Cons. Engr., Assoc. of Creosoting Companies of the Pacific Coast and West Coast Lumber Mfrs. Assoc., 909 White Bldg., Seattle, Wash	Assoc. Assoc. M.	Feb. 28, 1911 Dec. 2, 1914
HALL, HOWARD WASHINGTON. City Engr., Douglas, Ariz.		Dec. 2, 1914
HARRIS, GEORGE WILLIAM. Office Engr., Mackenzie, Mann & Co., Ltd., Port Arthur, Ont., Canada		Dec. 2, 1914
HINES, HOMER ESTLE. Pres., Hines Coal & Lumber Co., Princeton, W. Va	Jun. Assoc. M.	May 5, 1908 Oct. 7, 1914
HORTON, JOHN WILLIAM. Chf. Deputy County Surv., Sacramento County, 2614 California Ave., Sacramento, Cal	Jun. Assoc. M.	Sept. 1, 1908 Dec. 2, 1914
JONES, BENJAMIN EARL. Asst. Engr., U. S. Geological Survey, Water Resources Branch, 52 Montana Bank Bldg., Helena, Mont	Jun. Assoc. M.	April 6, 1909 Dec. 2, 1914
KNISKERN, LEWIS THAYER. 124 West 80th St., New York City	Jun. Assoc. M.	Feb. 6, 1912 Dec. 2, 1914
KRIEGER, ALBERT AUGUST. Res. Engr., Louisville Water Co., 1141 Cherokee Rd., Louisville, Ky	Jun. Assoc. M.	June 1, 1909 Dec. 2, 1914
LACEY, EDWARD PHILIP. City Engr., Bradentown, Fla		June 24, 1914
LYNCH, EDWARD MELVILLE. City Engr., City Hall, Glendale, Cal		Dec. 2, 1914
MCKIBBEN, HARRY SEYMOUR. 107 Mulberry St., Warren, Ohio		Dec. 2, 1914
MAGUIRE, GEORGE. 3703 Park Heights Ave., Baltimore, Md.		Dec. 2, 1914
MERRIMAN, LESLIE WILLIAM. Supt. of Constr., Am. Pipe & Constr. Co., 2721 West Lehigh Ave., Philadelphia, Pa		Dec. 2, 1914

ASSOCIATE MEMBERS (*Continued*)

	Date of Membership.
MILLARD, CHANCEY SPENCER. Mgr. and Chf. Engr., Francis S. Swales, Suite 49, Caroline Court, Vancouver, B. C., Canada	Dec. 2, 1914
MONETT, HARRY. Asst. Engr., Spring Val. Water Co., 375 Sutter St., San Francisco, Cal. — Jun.	Oct. 3, 1911
— Assoc. M.	Dec. 2, 1914
NOLAN, THOMAS BREW, JR. The Hawarden, 1419 R St., N. W., Washington, D. C	Dec. 2, 1914
PRINGLE, HERBERT LUTHER. Care, United Fruit Co., Santa Marta, Colombia	Sept. 2, 1914
REINHARDT, WILLIAM. Civ. and Hydr. Engr. (Hickey & Reinhardt), 422 Montgomery St., San Francisco, Cal	Dec. 2, 1914
REMINGTON, JOHN CLEMENT, JR. Borough Engr., 601 Market St., Camden, N. J	Dec. 2, 1914
RICE, PHILIP DURKEE. Engr., San Jose Water Co., 374 West Santa Clara St., San Jose, Cal	Dec. 2, 1914
RICHARDSON, ROBERT EARL. City Engr., Longmont, Colo	Dec. 2, 1914
SCHUTT, ALFRED GEORGE. Engr. and Estimator, Irwin & Leighton, Dime Bank Bldg., Detroit, Mich	Sept. 2, 1914
SOUTHWORTH, JAY LA DREW. Asst. Civ. Engr. in Chg., Lock Constr., Barge Canal, R. F. D. No. 5, Rome, N. Y	Dec. 2, 1914
SPIVEY, WILLIS TILLMAN. Gen. Supt., G. Lloyd Preacher, 415 Chronicle Bldg., Augusta, Ga	Dec. 2, 1914
THOMPSON, MORRIS. Engr. and Gen. Supt., Bates & Hudnall, Room 2, Gould Bldg., Tampa, Fla	Dec. 2, 1914
THOMSON, LESSLIE RIELLE. Designer, Dominion Bridge Co., Ltd., 90 Union Ave., Montreal, Que., Canada	Dec. 2, 1914
TUFTS, WILLIAM. 30 Beaufort Rd., Jamaica Plain, Mass. — Jun.	June 6, 1911
— Assoc. M.	Nov. 4, 1914
VON BLÜCHER, CARL FELIX KILL-MAR. Civ. Engr. and Surv. (C. F. H. von Blücher & Sons), Corpus Christi, Tex. — Jun.	Oct. 30, 1906
— Assoc. M.	Dec. 2, 1914
WACHTEL, LOUIS. Asst. Engr., State Highway Comm., Wells, N. Y. — Jun.	Mar. 2, 1909
— Assoc. M.	Sept. 2, 1914
WILBER, JOSEPH FREMONT. Asst. Engr., Hollis French & Allen Hubbard, 88 Pearl St., Boston, Mass	Dec. 2, 1914
WOODRUFF, CHARLES WILLIAM. 320 Henry Bldg., Portland, Ore. — Jun.	Jan. 4, 1910
— Assoc. M.	Dec. 2, 1914
WRIGHTSON, FRANCIS GERMAN, JR. Pres., Wrightson, Anderson & Yost, Room 307, California Fruit Bldg., Sacramento, Cal	Sept. 2, 1914
ZIEGLER, ADAM. Tenafly, N. J	Dec. 2, 1914

ASSOCIATE

	Date of Membership.
Stone, William Greene. 18 Mann Bldg., Utica, N. Y....	Dec. 2, 1914

JUNIORS

Alexander, Edward Porter. Engr. and Contr. (Alexander & Farrell), 418 Manhattan Bldg., Duluth, Minn....	Nov. 4, 1914
Anderson, Walter Seigfreid. 7417 Emerald Ave., Chicago, Ill.....	Dec. 2, 1914
Barzaghi, Arthur Jerome. Treas. and Secy., Barzaghi-Vought Co., Inc., 299 Madison Ave., New York City.	Dec. 2, 1914
Buell, William Elijah, Jr. 822 New Birks Bldg., Montreal, Que., Canada.....	June 24, 1914
Clark, Howard Foster. Asst. Engr., California R. R. Comm., 833 Market St., San Francisco, Cal.....	Dec. 2, 1914
Copeland, Robert Morris. Care, Interstate Commerce Comm., Div. of Valuation, Kansas City, Mo.....	Dec. 2, 1914
Crocker, Fred Wilbur. Engr., Pacific Bridge Co., 985 Front St., Portland, Ore.....	Dec. 2, 1914
Dakin, Robert Edward. Asst. Engr., The Connecticut Co., 330 Alden Ave., New Haven, Conn.....	Dec. 2, 1914
Gibbs, William Wetmore. Asst. Gen. Mgr., The Phosphate Min. Co., Nichols, Fla.....	Oct. 7, 1914
Krigbaum, Lowell Gaynor. Care, U. S. Engr. Office, 337 Eddy St., San Francisco, Cal.....	Nov. 4, 1914
Lynch, Conlaw Robert. In Chg., Newport Branch Office, B. Thomas Buffinton, Y. M. C. A., Mary St., Newport, R. I.....	Dec. 2, 1914
Miller, Karl Andrew. Instr. in Math. and Surveying, Allegheny Coll., 548 Baldwin St., Meadville, Pa....	Dec. 2, 1914
Ogden, Merton Miles. Balboa Heights, Canal Zone, Panama.....	Oct. 7, 1914
Roberts, Burke Brockway. Asst. Engr., James L. Stuart, 917 Illuminating Bldg., Cleveland, Ohio.....	Dec. 2, 1914
Waller, John Malcolm. Checker Draftsman, Kansas City Structural Steel Co., 3228 Morrell Ave., Kansas City, Mo.....	Dec. 2, 1914
Whitney, Charles Smith. 704 Stewart Ave., Ithaca, N. Y.....	Dec. 2, 1914
Wong, Jick Gam. Senior Asst. Engr., Szechuen-Hankow Ry., Ichang Ry. Station, Ichang, China.....	Oct. 7, 1914

RESIGNATIONS

MEMBERS

	Date of Resignation.
Corner, Charles.....	Dec. 31, 1914
Delano, Frederic Adrian.....	Dec. 31, 1914
Frye, Albert Irvin.....	Dec. 31, 1914

MEMBERS (*Continued*)	Date of Resignation.
Hutchinson, Cary Talcott	Dec. 31, 1914
Jarvis, Charles Maples	Dec. 31, 1914
Kenney, Edward Fulbister	Dec. 31, 1914
Leidy, George Craig	Dec. 31, 1914
Mills, Charles Malon	Dec. 31, 1914
Post, Henry Willis	Dec. 31, 1914
Schulze, Henry Atherton	Dec. 31, 1914
Tone, Sumner La Rue	Dec. 31, 1914
Walker, Frank Hiram	Dec. 31, 1914
Wood, George	Dec. 31, 1914
ASSOCIATE MEMBERS	
Andress, Harry R.	Dec. 31, 1914
Brenn, Charles Ferdinand	Dec. 31, 1914
Cadle, Charles Longford	Dec. 31, 1914
Cooper, Kenneth Farra	Dec. 31, 1914
Davis, Arthur Albert	Dec. 31, 1914
Fenton, Louis Gillespie	Dec. 31, 1914
Kingman, John Jennings	Dec. 31, 1914
Kirby, William Franklin	Dec. 31, 1914
Koch, John Christian	Dec. 31, 1914
Martin, Walter Irving	Dec. 31, 1914
Newman, Arthur Traver	Dec. 7, 1914
Shepard, Edward Lewis	Dec. 31, 1914
Stevens, Perley Egbert	Dec. 31, 1914
Tyler, William Rogers	Dec. 31, 1914
Welton, Benjamin Franklin	Dec. 31, 1914
ASSOCIATES	
Currier, Charles Gilman	Dec. 31, 1914
Marsh, Albert Lebeaux	Dec. 31, 1914
Smith, Cameron C.	Dec. 31, 1914
JUNIORS	
Akerly, Harold Edward	Dec. 31, 1914
Andrus, Leonard Alexander	Dec. 31, 1914
Cary, Richard Lucius	Dec. 31, 1914
Duryea, Robert Francis	Dec. 31, 1914
Lee, Chester Sherman	Dec. 31, 1914
Lincoln, Edward Lewis	Dec. 31, 1914
MacIlvaine, Francis Shippen	Dec. 31, 1914
MacKrell, Edwin Allan	Dec. 31, 1914
McCaffrey, John William	Dec. 31, 1914
Paine, George Hebard	Dec. 31, 1914
Rossig, Edgar William	Dec. 31, 1914

JUNIORS (*Continued*)	Date of Resignation.
Sims, Harry Hillyer	Dec. 31, 1914
Stewart, William James	Dec. 31, 1914
Tillit, Pedro Ernesto	Dec. 31, 1914
Wilson, Claude Thomas	Dec. 31, 1914
Wilson, Rodney Melledge	Dec. 31, 1914

DEATHS

Blatchford, Eliphalet W. Elected Fellow, February 6th, 1873; died January 25th, 1914.

Eckart, William Roberts. Elected Member, January 5th, 1881; died December 8th, 1914.

Greene, Robert Maxson. Elected Associate Member, June 1st, 1898; Member, September 1st, 1908; died December 5th, 1914.

Hufschmidt, William Paul. Elected Associate Member, October 4th, 1910; died August 13th, 1914.

King, Wallace, Jr. Elected Associate, December 4th, 1900; died October 31st, 1914.

Rood, Henry Martyn. Elected Member, December 3d, 1890; died November 24th, 1914.

Steere, Edmund Job. Elected Member, October 1st, 1913; died July 18th, 1914.

Total Membership of the Society, January 7th, 1915, 7 678.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(December 2d, 1914, to January 1st, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Steel and Iron,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 20c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.

LIST OF ARTICLES

Bridges.

St. Paul Street Bridge, St. Catharines.* H. C. Taylor. (96) Nov. 26.
Metropolitan Railway Development, New Viaduct at Kilburn.* (12) Nov. 27.
Reinforced Concrete Bridges. Daniel B. Luten. (Paper read before the Kansas Eng. Soc.) (67) Dec.
Plant and Working Methods on Approaches to North Side Point Bridge.* E. L. Jones. (13) Dec. 3.
Construction of the Gwynn's Falls Arch Bridge.* (15) Dec. 4.
Design of the Main Shoes of the New Quebec Bridge.* (12) Dec. 4.
Reinforced-Concrete Bridge with Cantilever Abutments.* (14) Dec. 5.
Building Bridges Under Fire.* Martin Wells. (46) Dec. 5.
Design, Construction and Detailed Costs of the Richelieu River Bridge, Lacolle Junction, Quebec.* (86) Serial beginning Dec. 9.
Some Bridges on the Columbia Highway.* K. P. Billner. (13) Dec. 10.

*Illustrated.

Bridges—(Continued).

Steel Arch Bridge on Curved Gradient in the Bietsch Valley.* **(12)** Dec. 11.
Steel Bridge Standards of the Iowa Highway Commission.* E. F. Kelley. **(14)** Dec. 12.
Details of Main Span of Detroit-Superior Bridge at Cleveland.* **(14)** Dec. 12.
Rosedale Section, Bloor Street Viaduct, Portion No. 2 of the Proposed $2 500 000 Structure to Bridge the Don and Rosedale Valleys in Northeast Toronto.* **(96)** Dec. 17.
Erecting Railroad Viaduct Superstructure.* **(14)** Dec. 19.
Concrete Handrails on Kansas City Terminal Work, Unit Construction with Spindles, Posts and Rails, Cast at a Central Plant, Used in Protecting 1¾ Miles of Viaducts.* **(14)** Dec. 19.
End Supports of Lachine Bridge Spans.* **(14)** Dec. 19.
From the Soil Up, a New Method of Designing.* Arvid Reuterdahl. **(86)** Dec. 23.
Reinforced-Concrete Highway Arch for Grade Separation.* **(14)** Dec. 26.
Chief Features in Building a Long Concrete Viaduct at St. Louis; Experiences in Driving Concrete Piles with Two Types of Hammers and Costs of Some Items in Structure Spanning Thirty-five "Live" Tracks.* P. A. Richardson. **(14)** Dec. 26.
Mud Lake Bridge Substructure; Piers were Sunk to Maximum Depth of Nearly 104 Ft. by Combination of Open-Dredging and Pneumatic-Caisson Processes.* **(14)** Dec. 26.
Fabricating Steelwork for the Hell Gate Arch; Methods Employed at the Ambridge Shops for the Manufacture of the Principal Members of the 977½-Foot Spandrel-Braced Arch Trusses.* **(14)** Dec. 26; **(13)** Dec. 3.
Méthode Expéditive pour le Calcul des Voutes.* Mesnager. **(33)** Dec. 1.
Grossere ausgeführte Gelenkbrücken in Eisenbeton.* F. Baumstark. **(51)** Serial beginning Sup. No. 22.
Amerikanische Zerreissversuche mit Grossen Augenstäben. J. Melan. **(69)** Oct.
Wirtschaftliche Begründung der Abmessungen von Blechträgern welche einem Kosten-kleinstwert Entsprechen. G. Böheim. **(69)** Serial beginning Oct.
Eisenbetonstrassenbrücke bei Songavazzo (Bergamo).* **(78)** Nov. 6.
Die Lastenförderung durch Kabelkrane, insbesondere beim Bau der Camsdorfer Brücke.* Hans Schäfer. **(78)** Nov. 6.

Electrical.

Over-Pressure Protective Gear for High-Tension Circuits.* Kenelm Edgcumbe. **(26)** Serial beginning Nov. 27.
The Telephone Stations of the World at the Beginning of 1913 (with Some Statistics Relating to 1914). W. H. Gunston. (Abstract from *Telegraph and Telephone Journal.*) **(73)** Nov. 27.
An Artificial Equivalent of an Open-Wire Line for Telephonic Experiments. G. M. B. Shepherd. (Paper read before the Post Office Elec. Engrs.) **(73)** Nov. 27.
Magnet-Switch Crane Controllers.* H. H. Broughton. **(73)** Serial beginning Nov. 27.
Effect of Altitude on the Spark-Over Voltages of Bushings, Leads and Insulators.* F. W. Peek, Jr. **(42)** Dec.
Insulator Depreciation and Effect on Operation.* A. O. Austin. **(42)** Dec.
Electricity in the Lumber Industry.* E. F. Whitney. **(42)** Dec.
Mine Motors. Everett Drennen. **(45)** Dec.
Temperature Rise in Twin Flexible Wires.* S. W. Melsom. **(77)** Dec. 1.
Experiments on the Heating of Screw-Socket Lampholders.* Clifford C. Paterson. **(77)** Dec. 1.
The Hackney Electricity Works Extensions.* **(73)** Dec. 4; **(26)** Dec. 4.
Costs of Electric Shovel Work at Cleveland.* **(17)** Dec. 5.
Distributing Potential Over a String of Insulators.* J. L. Brenneman and Harold M. Crothers. **(27)** Dec. 5.
Characteristics of Crystal Rectification.* Alan E. Flowers. (Abstract from *Physical Review.*) **(73)** Dec. 11.
The Magnetic Field of the Three-Phase Induction Motor.* F. T. Chapman. **(73)** Dec. 11.
New Portland Station of American District Telegraph Co.* C. M. Marron. **(111)** Dec. 12.
Operating Features of the Audion.* E. H. Armstrong. **(27)** Dec. 12.
Long Transmission Lines.* R. A. Philip. **(27)** Dec. 12.
Cables.* C. J. Beaver. **(77)** Dec. 15.
The Power-Factor Indicator.* John A. Randolph. **(64)** Dec. 15.
Detroit's Municipal Lighting Plant.* Thomas Wilson. **(64)** Dec. 15.
Recent Advances in Electrical Crane and Lift Design.* **(26)** Serial beginning Dec. 18.
Electricity in the Printing Office.* Frank Broadbent. (Abstract of paper read before the Assoc. of Engrs.-in-Charge.) **(11)** Dec. 18.
Limitations in the Construction of High-Tension Direct-Current Machines.* A. Scherbius. (Abstract from *Elektrische Kraftbetriebe und Bahnen.*) **(73)** Dec. 18.
The Possibility of Sharp Directive Wireless Telegraphy.* E. Bellini. **(73)** Dec. 18.

*Illustrated.

Electrical—(Continued).

Extensions at Lister Drive Station, Liverpool.* (73) Serial beginning Dec. 18.
Characteristics of the Three-Wire Generator.* O. J. Ferguson. (27) Dec. 19.
Motors for Ventilating and Pumping Systems.* C. E. Clewell. (72) Dec. 24.
Photometry of the Gas-Filled Lamp.* G. W. Middlekauff and J. F. Skogland. (27) Dec. 26.
Electrical Equipment of Panama-Pacific Exposition.* (27) Dec. 26.
Calculations of Starting Resistances for Railway Motors.* F. Castiglioni. (17) Dec. 26.
Automatic Telephone System at the Kansas City Terminal.* (18) Dec. 26.
Power Plant of the Acme Tea Company.* (64) Dec. 29.
High-Voltage Transmission at High Altitude.* Percy H. Thomas. (27) Serial beginning Jan. 2.
Tafeln und Tabellen zum schnellen Bestimmen elektrischer Leitungen.* Th. Vaillant. (41) Nov. 19.
Städtische Momentreserven für elektrische Beleuchtung mit besonderer Berücksichtigung der Verhältnisse in Zürich.* W. Kummer. (107) Serial beginning Nov. 21.
Eine neue Effektbogenlampenkohle (Crustakohle).* H. G. Hatfield. (41) Nov. 26.
Ueber ein Ohmmeter mit mehreren Messbereichen.* H. A. W. Klinkhamer. (41) Nov. 26.
Belastungsmessungen in Gleichstromnetzen.* L. Lewin. (41) Dec. 3.

Marine.

Failure of Steel Ship-Plates.* W. J. B. Wilson. (Paper read before the North-East Coast Institution of Engrs.) (11) Dec. 11; (47) Dec. 4.

Mechanical.

Test of Large Reversing Engine and Rolling Mill.* Karl Nibecker. (58) July.
Arithmetical Machines.* H. E. Goldberg. (4) Nov.
Woodall-Duckham Vertical Retorts at Bolton.* Robert Neill. (Paper read before the Manchester and District Junior Gas Assoc.) (66) Nov. 10.
Gas as an Illuminant. Malcolm Littlejohn. (Paper read before the Scottish Junior Gas Assoc.) (66) Nov. 10.
Coal and Coke Handling Machinery at the New South Amsterdam Gas-Works.* (66) Nov. 24.
Darlington Solely Supplied with Gas from Continuous Verticals.* (66) Nov. 24.
Four-Ton Petrol Motor Lorry.* (12) Nov. 27.
The Field of the Commercial Electric Battery Vehicle. W. E. Warrilow. (Abstract of paper read before the Wolverhampton District Eng. Soc.) (73) Nov. 27.
Gas and Electric Rates at Long Eaton and Ilford. Norton H. Humphrys. (24) Nov. 30.
Gas Rates, Flat-Rate Unfair to Company and Consumer. J. E. Bullard. (24) Nov. 30.
Purifier Installations.* C. E. Paige. (Paper read before the Am. Gas Inst.) (24) Nov. 30.
Cooling Towers and Cooling Ponds. Walter G. Stephan. (Paper read before the Ohio Soc. of Elec., Mech., and Steam Engrs.) (62) Nov. 30.
The Briquetting Plant.* Clancy M. Lewis. (45) Dec.
Bituminous Coal Storage.* (45) Dec.
Wing Data and Analysis for a Staggered Biplane.* A. F. Zahm. (3) Dec.
Gas in Honolulu. H. L. Strange. (Paper read before the Pacific Coast Gas Assoc.) (83) Dec. 1.
The Universal Unaflow Engine.* (64) Dec. 1.
Performance of Low-Pressure Turbine Plant.* Charles H. Bromley. (64) Dec. 1.
Separation of the Illuminants in Mixed Gas. G. A. Burrell and I. W. Robertson. (Paper read before the Am. Gas Inst.) (83) Dec. 1.
Lead Acetate Test for Hydrogen Sulphide in Gas.* R. S. McBride and J. D. Edwards. (Abstract from *Technologic Paper No. 41*, U. S. Bureau of Standards.) (83) Dec. 1.
Gas Piping Specifications and Inspection. A. E. Turner. (Paper read before the Am. Gas Inst.) (83) Dec. 1; (101) Dec. 11.
A System for Long Distance Transmission of Readings of Indicating and Recording Instruments.* (13) Dec. 3.
The Respeeding of Lathes.* L. P. Alford. (72) Dec. 3.
Sand Mixing Plant for a Large Foundry.* (20) Dec. 3.
Internal-Combustion Engines from a Commercial Standpoint.* James G. Walthew. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Dec. 4.
Washing, Coking and By-Product Plant at Coedely Colliery.* (22) Dec. 4.
A Uniflow Undertype Engine.* (12) Dec. 4.
Gas Rates, Specific Examples of the Benefits from Scientific Rates, Suggests a Rate Based on Use, not on Quantity Used. H. L. Coleman. (Paper read before the Empire State Gas and Elec. Assoc.) (24) Dec. 7.

*Illustrated.

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Testing of Steel Motor Truck Wheels.* (62) Dec. 7.
Transmitting Power by Rope Drives.* Henry D. Jackson. (64) Dec. 8.
Experiences in Small Gas-Works. Alexander Wright. (Paper read before the Scottish Junior Gas Assoc.) (66) Dec. 8.
Equipment and Methods in Largest Refrigeration System.* Charles H. Bromley. (64) Serial beginning Dec. 8.
Winslow High-Pressure Boiler.* (64) Dec. 8.
The Corrosion of Refractories by Salts in Coal. Alfred B. Searle. (66) Dec. 8.
Handling Construction Material by Aerial Tramways.* (13) Dec. 10.
The Respeeding of Machine Tools.* L. P. Alford. (72) Dec. 10.
Chain Gearing.* (47) Dec. 11.
Handling Freight with Storage Battery Trucks.* (15) Dec. 11.
New Type of Traveling Excavator for Ditches.* W. W. Patch. (14) Dec. 12.
The Removal of Carbon Bisulphide from Coal Gas. E. V. Evans. (Paper read before the Soc. of Chemical Industry.) (66) Dec. 15.
A New Vertical Retort Installation at Ossett.* (66) Dec. 15.
Standard Methods of Gas-Testing in America. (From *Circular No. 48*, U. S. Bureau of Standards.) (66) Dec. 15.
Efficiency of Commercial Pipe Covering.* L. B. McMillan and H. S. Rekersdres. (From *Wisconsin Engineer.*) (101) Dec. 18.
Jigs.* Max R. Lawrence. (Paper read before the Manchester Assoc. of Engrs.) (12) Dec. 18.
The Limits of Efficiency of Gas-Engines.* Thomas B. Morley. (11) Dec. 18.
Gas Manufacture from the Point of View of Physical Chemistry. W. F. Rittman. (Paper read before the Am. Gas Inst.) (24) Dec. 21.
Tests of a New Recording Calorimeter.* C. H. Stone and W. H. Hinman. (Paper read before the Am. Gas Inst.) (24) Dec. 21.
A Detroit Flour-Mill Plant.* Thomas Wilson. (64) Dec. 22.
Republic Company's By-Product Coke Plant.* (20) Dec. 24.
The Mechanical Drying of Ore Products.* C. O. Bartlett. (82) Dec. 26.
The Balance of Revolving Bodies.* F. A. Halsey. (72) Serial beginning Dec. 31.
The European War from an Engineer's Standpoint, the Conquest of the Air and Its Results on Warfare.* John B. C. Kershaw. (9) Jan.
Eine Neuzeitliche Haldenbeschickungsbahn.* Brugsch. (69) Sept.
Die Bedeutung der Elektromobile für den Stromabsatz der Elektrizitätswerke.* H. Beckmann. (41) Serial beginning Nov. 12.
Die Giesserei der Ford Motor Co., Detroit.* E. Leber. (50) Nov. 26.

Metallurgy.

The Interstrain Theory of Hardness.* Andrew McCance. (Paper read before the Faraday Soc.) (11) Nov. 27.
The Rennerfelt Electric Furnace.* (11) Nov. 27.
Cyanidation of Clear Creek and Gilpin County Sulphides.* Jackson A. Pearce. (103) Nov. 28.
Proposed Regulations for Blast Furnaces.* F. H. Willcox. (Paper read before the Industrial Welfare and Efficiency Conference.) (62) Nov. 30.
The New Zinc Smeltery at Langeloth.* W. R. Ingalls. (16) Dec. 5.
Spelter Comes to the Steel; Two New Modern Zinc Plants Go in Operation Simultaneously. (62) Dec. 7.
New Smelting Works of the Mond Nickel Co.* (16) Dec. 12.
Report of the Selby Smelter Commission. (Concerning the Alleged Effects Produced by Smoke.)* (103) Dec. 19.
Pulp Constants.* G. H. Clevenger, H. W. Young and T. N. Turner. (16) Dec. 19.
Small Mill and Reduction Plant for Mill Tests.* (82) Dec. 19.
Operations and Costs at the Motherlode Mill. W. P. Anderson. (82) Dec. 19.
Concentrate Treatment Costs at the Treadwell Cyanide Plant.* W. P. Lass. (103) Dec. 19.
Handling Iron and Cinder at the Blast Furnace.* J. E. Johnson, Jr. (105) Serial beginning Jan.
Comparisons of Electric Resistivities at High Temperatures.* Carl Hering. (105) Jan.
Precipitating Plant at the Copper Queen Mines.* Charles M. Coats and Glen L. Allen. (16) Jan. 2.
The Copper Queen Smelting Works.* Richard H. Vail. (16) Jan. 2.
Neue Gesichtspunkte zur Beurteilung des Kohlenverbrauchs in Zementdrehrohröfen. Otto Dormann. (80) Oct. 24.
Neuanlagen von Hüttenwerken in Amerika.* (50) Serial beginning Nov. 5.
Metallbriketts. E. F. Hirsch. (41) Dec. 3.

Military.

Rifles and Mortars.* (46) Dec. 5.
Field Gun and Aerial Projectiles.* (46) Dec. 5.
Mining and Countermining of Fortifications.* (46) Dec. 5.
Artillerie Lourde Française.* A. Dumas. (3) Serial beginning Dec. 1.

*Illustrated.

Mining.

Notes on Coal-Mining in the State of Illinois, United States of America.* Samuel Dean. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 48, Pt. 1.

Hydraulic Stowing in the Gold-Mines of the Witwatersrand.* B. C. Gullachsen. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 48, Pt. 1.

An Instrument for the Detection of Firedamp in Mines.* F. J. Turquand. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 48, Pt. 1.

Tests on False (or Split) Links for Cut-Chain Braes.* John T. Wight. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 48, Pt. 1.

A Portable Electrical Gas-Detecting Device for Use with Miner's Lamps.* George J. Ralph. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 48, Pt. 1.

The Rosehall Signal-Indicator.* James Black. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 48, Pt. 1.

Miners' Electric Lamps Compared with the Combustion-Tube Oil-Lamp.* E. A. Hailwood. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 48, Pt. 1.

The Testing of Fans with Special Reference to the Measurement of Pressure.* Thomas Bryson. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 48, Pt. 1.

Application of Jigs to Gold Dredging.* J. W. Neil. (103) Nov. 28.

Mining Methods Used in Copper Queen Mine.* Joseph Park Hodgson. (82) Nov. 28.

Underhand Square Setting for Finishing Stopes.* Andrew Fairweather. (Abstract of paper read before the Australasian Inst. of Min. Engrs.) (16) Nov. 28.

Stripping with Drag-Line Excavators.* L. E. Ives. (16) Nov. 28.

Rock Work in Coal Mines.* (45) Dec.

The Royalton Mine Explosion.* (45) Dec.

English Coal Washing Plant.* (45) Dec.

The J. P. Karns Heading Machine.* (36) Dec.

The Phosphate Rock Industry of Florida.* Lester W. Tucker. (109) Dec.

The Prevention of Electrical Accidents in Mines.* T. J. Nelson. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Dec. 4.

Investigations into a System of Electric Bell Signalling for Use in Collieries.* Thomas G. Watts. (57) Dec. 4.

Explosion Test at the Experimental Mine.* George S. Rice and L. M. Jones. (103) Dec. 5.

Report on the Wharncliffe Silkstone Explosion.* (57) Dec. 11; (22) Dec. 11.

Water-Dams at Netherseal Colliery.* C. Dickinson. (22) Dec. 18.

The Safe Use of Electricity in Coal Mining. W. M. Thornton. (Paper read before the British Assoc.) (19) Dec. 19.

The Stripping, Quarrying and Mining of Gypsum. L. H. Cole. (From Report to the Dept. of Mines of Canada.) (96) Dec. 24.

Examination of Placer Ground.* Thomas A. Graves. (103) Dec. 26.

Kubas Eisenerzbergbau.* Bror Orton. (50) Nov. 19.

Miscellaneous.

An Empirical Formula for the Design of Retaining Walls.* C. P. Symonds. (96) Nov. 26.

Development of Scientific Illumination. F. E. Cady. (Paper read before the Illuminating Eng. Soc.) (83) Dec. 1.

The Crushing Strength of Ice. (96) Dec. 10.

A Discussion of Some Legal Principles of Interest to Engineers. William L. Bowman. (Paper read before the Harvard Eng. Soc.) (86) Dec. 16.

Artificial Daylight, Its Production and Use. M. Luckiesh and F. E. Cady. (Paper read before the Illuminating Soc.) (24) Dec. 21.

The Industry of the Coal Tar Dyes, An Outline Sketch. Bernhard C. Hesse. (From *Journal of Industrial and Eng. Chemistry.*) (24) Dec. 28.

The Hardwood Distillation Industry in America.* Edward H. French and James R. Withrow. (Paper read before the Am. Inst. of Chemical Engrs.) (105) Jan.

Municipal.

Some Notes on Doors for Manholes and Mudholes.* J. R. Edwards. (47) Serial beginning Nov. 27.

Improved Road Surfacing in Grangemouth.* David A. Donald. (104) Nov. 27.

New Municipal Asphalt Plant at Pittsburgh, Pa.* Norman S. Sprague. (60) Dec.

Bituminous Macadam Roads in Rhode Island. Irving W. Patterson. (Paper read before the Atlanta Road Congress.) (60) Dec.

A Playground for a Small City.* Warfield Webb. (60) Dec.

Unit Costs of Bituminous Carpet Treatment and Bituminous Macadam by the Penetration Method on Two Roads in Illinois. (86) Dec. 2.

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The Location and Width of Highways and the Securing of Rights of Way. Austin B. Fletcher. (Paper read before the Am. Road Congress.) (86) Dec. 2.
Road Economics. J. E. Pennybacker. (Paper read before the Am. Road Congress.) (96) Dec. 3; (86) Dec. 2.
Road Building with Industrial Equipment.* (14) Dec. 5.
Handling 400 Tons of Stone per Day with Auto Trucks. (14) Dec. 5.
Oneida Street Crossing under Milwaukee River.* (14) Dec. 5.
Methods and Cost of Resurfacing Water Bound Macadam with Rocmac Macadam. J. H. Jackson. (From Annual Report of the Queen Victoria-Niagara Falls Park.) (86) Dec. 9.
Unit Costs of Resurfacing Portions of the Niagara River Road in Queen Victoria Park, Canada.* (86) Dec. 9.
Hauling Gravel with Motor Truck and Trailers: A Service Test in Concrete Road Construction.* (86) Dec. 9.
Some Practical Notes on Brick Road Construction. James M. McCleary. (Paper read before the Northwestern Road Congress.) (86) Dec. 9.
Methods and Cost of Removing an Asphaltic Macadam Road Surface, Reworking the Old Material and Relaying It as Asphaltic Concrete.* G. C. Dillman. (86) Dec. 9.
A Heavy Cut for a Street at Kansas City.* (13) Dec. 10.
Pavement Problems; Contrasts of American and Foreign Practice. Henry Welles Durham. (Abstract of paper read before the Am. Road Congress.) (13) Dec. 10.
Pavement Problems and Experience in San Francisco.* James M. Owens. (13) Dec. 10.
Improvement of Ordinary Country Roads. S. P. Hooker. (From paper read before the Am. Road Congress.) (96) Dec. 10.
An Automatic "Traffic Squad" at Lawrence.* (14) Dec. 12.
Practical Hints on Proper Methods of Maintenance for Concrete Pavements.* William M. Kinney. (14) Dec. 12.
Maintaining Macadam Streets in Kansas City; Experience with Pure Asphaltic Oils and Continuous Maintenance Contracts. Clark R. Mandigo. (14) Dec. 12.
Expenditures for Permanent and Perishable Features of Roads; Percentages of Total Cost which Should be Applied to Foundations, Drainage Structures, Alignment and Grades. (14) Dec. 12.
Thin Asphalt Block Pavement for New York State Highways.* Lowell Grossman. (14) Dec. 12.
Ideals in City Planning. Harrington Emerson. (Abstract from paper read before the Civic League of Cleveland.) (19) Dec. 12.
The Last Word in Brick Pavement.* (76) Dec. 15.
Standard Small Culverts Recommended by the Illinois Highway Commission.* (86) Dec. 16.
Methods and Cost of Constructing a Bituminous Carpet on Concrete Roads, with Notes on General Maintenance. F. H. Whitlow. (86) Dec. 16.
Location and Construction of Highways in Mountain Country.* F. W. Harris. (13) Dec. 17.
New Asphalt Plant at Montreal.* Daniel J. Hauer. (96) Dec. 17.
Some Requirements of Road Specifications with Special Reference to Rolling. W. W. Crosby. (Paper read before the Am. Assoc. for the Advancement of Science.) (86) Dec. 23.
Maintenance and Repair of Pennsylvania State Roads. (86) Dec. 23.
Patching Sheet Asphalt with Bituminous Concrete. (86) Dec. 23.
Bituminous Macadam by the Cold Mixing Method. Irving W. Patterson. (Paper read before the Atlanta Road Congress.) (96) Dec. 24.
Wood-Block Expansion Joint for Concrete Alley Pavements; Built-Up Strip Used in Baltimore to Protect Joints of Concrete Slabs in Narrow Roadways.* (14) Dec. 26.
Pipe Subways for the Public Utilities of Chicago. Louis Dumond. (Abstract of Report to Commission on Downtown Municipal Improvements.) (14) Dec. 26.
Dry Masonry Walls for Highway Embankments.* (14) Dec. 26.
Traffic and the Methods and Cost of Road Maintenance in Massachusetts and a Comparison with English and French Conditions. W. D. Sohier. (Paper read before the Am. Road Congress.) (86) Dec. 30.
Widening a Business Street at St. Paul, Minn.* (13) Dec. 31.
Repairing and Resurfacing Bituminous Pavements.* Samuel H. Lea. (13) Dec. 31.
How Small Communities May Have Good Roads. Logan Waller Page. (46) Jan. 2.

Railroads.

Railway Classification Yard Lighting. D. P. Morrison. (58) Oct.
The Delivery and Handling of Miscellaneous Freight at the Boston Freight Terminals.* Harold Pender, H. F. Thomson and C. P. Eldred. (65) Oct.
The Locomotive Stoker. J. C. F. Street. (61) Oct. 20.
Thirty Years of Locomotive Progress in Egypt. E. L. Ahrons. (12) Nov. 27.
New Union Passenger Station at Kansas City, Mo.* (18) Nov. 28.

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Electrified Terminal of the B. & O. R. R. at Baltimore.* J. H. Davis. (18) Nov. 28.
Progress on the Newark Terminal.* (17) Nov. 28.
The Parcel Post and Its Effect on Railway Revenues. V. J. Bradley. (65) Dec.
Painting Locomotives and Steel Cars. Milton L. Sims. (Paper read before the Central Ry. Club.) (25) Dec.
Jersey Central Steel Passenger Cars.* (25) Dec.
Baltimore & Ohio Milk Refrigerator Car.* (25) Dec.
Locomotive Front Ends, 1853-1913.* C. T. Rommel. (25) Dec.
Most Powerful Pacific Type Locomotive.* (25) Dec.
Express Refrigerator Cars; Illinois Central R. R.* (21) Dec.
0-6-2 Tank Engines; Taff Vale Railway.* (21) Dec.
Design of an Automatic Train Stop.* W. J. Zabel. (36) Dec.
The Parana Railway of Brazil.* F. E. Lawrence. (36) Dec.
A Problem in Economics of Mountain Railway Location.* J. G. Sullivan. (36) Dec.
Notes on Catenary Construction of New York, Westchester and Boston Railway. Sidney Withington. (3) Dec.
An English Railway Trainshed.* (13) Dec. 3,
Alloy-Steel Rails. (13) Dec. 3.
Passenger Locomotives for the Great Northern.* (15) Dec. 4.
Piercing the Selkirk Mountains for a Five-Mile Tunnel.* (14) Dec. 5; (15) Dec. 11.
Rail Sections as an Element in Steam and Electric Traction.* P. H. Dudley. (From *General Electric Review.*) (19) Serial beginning Dec. 5.
Improvements at the Englewood Locomotive Terminal and Car Repair Plant, L. S. & M. S. Ry., Chicago.* (18) Dec. 5.
A New Cast-Steel Clamp for Guard-Rails.* (13) Dec. 10.
The Railway Problem and Its Solution. Samuel Rea. (Paper read before the Chamber of Commerce.) (15) Dec. 11.
Failures or Breakdowns of Locomotives.* Robert Weatherburn. (12) Serial beginning Dec. 11.
Instructions for Roadway and Track Men on Federal Valuation; Abstract of Second Tentative Draft Recently Issued by Director of Valuation, Interstate Commerce Commission. (14) Dec. 12.
Scrap-Handling Plant of Boston and Albany Railroad.* (14) Dec. 12.
Pacific Type Locomotives, Chesapeake & Ohio Railway.* (18) Dec. 12.
Tunnel and Terminal Electrification of the Canadian Northern Ry.* W. C. Lancaster. (18) Dec. 12.
An Ambulance Train on an English Railway.* (13) Dec. 17.
Rail Joints and Tie Spacing.* (13) Dec. 17.
Tunnel Lining by Compressed Air.* (15) Dec. 18.
Extensive Electrification on the St. Paul.* (15) Dec. 18.
Heavy 4-8-2 Locomotive for South African Railways. (12) Dec. 18.
New American Tunneling Record at Rogers' Pass; Progress of 817 Feet was Made Through Slate Material during Month of November on Canadian Pacific Railway's Work. (14) Dec. 19.
Looping the Loop in the Alps.* J. F. Springer. (46) Dec. 19.
The Operation of Large Yards. A. M. Umshler. (From *Illinois Central Magazine.*) (18) Dec. 19.
Ventilation of Allegheny Summit Tunnel, Virginian Ry.* F. F. Harrington. (From *General Electric Review.*) (18) Dec. 19.
New Passenger Station of the D., L. & W. R. R. at Morristown, N. J.* (18) Dec. 19.
Operating Plans for the Electrified Division of the C. M. & St. P.* (17) Dec. 19.
Two Pacific Type Locomotives of High Power.* (15) Dec. 25.
Automatic Signals on the Lehigh & Hudson River.* (15) Dec. 25.
The Decision Granting the Five Per Cent. Rate Advance. (15) Dec. 25.
Pneumatic Track Tamping, Portable Outfits in Use on New York Central & Hudson River Railroad.* (14) Dec. 26.
Pacific Type Locomotives for the Delaware & Hudson Co.* (18) Dec. 26.
Electrification of London & Port Stanley Railroad.* (17) Dec. 26.
Frog Guard Rails of Special Design.* (13) Dec. 31.
L'Electrification du Chemin de Fer de Luleo à Narvik (Suède et Norvège).* (33) Dec. 1.
Das Eisenbahnverkehrswesen auf der Weltausstellung in Gent 1913. Guillery. (102) Serial beginning Oct. 1.
Die Ursachen der Schlaglochbildung an den Radreifen der Lokomotiven.* J. Jahn. (102) Oct. 1.
Zum Durchschlag des Grenchenbergtunnels.* (107) Nov. 21.
Elektrische Beleuchtung von Personenwagen nach dem vereinfachten System Dick.* Emil Dick. (41) Serial beginning Nov. 26.
Wechselstrom-Triebwagen auf der Strecke Spiez-Frutigen der Lötschbergbahn.* (41) Nov. 26.
Wirkungsweise der Zugbeleuchtungsanlage.* (41) Dec. 3.

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Pressed-Steel Cars for the Chicago Elevated.* (17) Dec. 5.
Reinforced Concrete Poles for Railways.* (17) Dec. 5.
New Cars of Seattle Municipal Railway.* H. J. Kennedy. (17) Dec. 12.
Analysis of Stresses in Chicago Elevated Steel Car.* H. A. Johnson. (17) Dec. 12.
A Novel Portable Substation for the Berkshire Street Ry.* (13) Dec. 17.
The New York Municipal Car, Trucks, Brake, Rigging and Draft Gear.* (17) Dec. 26.
Die Kabelführung städtischer Schnellbahn.* (41) Nov. 12.

Sanitation.

Design of Two Residential Sewage Treatment Plants, Including Settling Tanks of Imhoff Type.* Samuel A. Greeley. (86) Sept. 16.
Sewage Disposal Works at Leeds.* (12) Nov. 27.
Experimental Plant for Treating Sewage at Brooklyn, N. Y.* George T. Hammond. (60) Dec.
Concrete Septic Tanks.* (67) Dec.
Electric Cooking and Heating in Private Houses.* W. A. Gillott. (77) Dec. 1.
New Sewage Pumping, Screening and Sterilizing Station at Daytona, Fla.* (86) Dec. 2.
Some Possibilities in Ventilation.* A. H. Barker. (Paper read before Univ. College.) (11) Dec. 4.
Construction Methods and Plant for Diversion and Drainage Works; Forty-Three Million Yards of Ditch and Levee Work for Little River Drainage District in Missouri.* (14) Dec. 5.
Investigation of Sewer Air Following Boston Explosion. H. W. Clark. (14) Dec. 5.
Construction of the Metcalf Ave. Sewer, Borough of the Bronx, New York City.* G. L. Christian. (13) Dec. 10.
Draining Land by Wells; Concrete Drain Head.* (13) Dec. 10.
A Comparison of the Economy of Powdered Coal, Oil and Water Gas for Heating Furnaces. C. F. Herington. (13) Dec. 10.
Construction of Concrete Septic Tanks.* Percy H. Wilson. (101) Dec. 11.
Aeration Experiments with Activated Sludge. W. H. Duckworth. (Paper read before the Assoc. of Managers of Sewage Disposal Works.) (104) Dec. 11.
Gas-Heated Hot-Water Supplies.* W. B. Cobb. (Paper read before the Midland Junior Gas Assoc.) (66) Dec. 15.
Construction Plant and Methods Employed on the North Shore Intercepting Sewer, Sanitary District of Chicago.* M. A. Berns. (86) Dec. 16.
Recommended General Procedure in Sewage Works Operation.* (Report of Committee of the Am. Public Health Assoc.) (86) Dec. 16; (13) Dec. 17.
Sewage Intercepting Chamber and Settling Tank for a Public Institution at Geneva, Ill.* (13) Dec. 17.
Plumbing Equipment in Mercantile Building.* (101) Dec. 18.
A Recent Development of the Sewage Problem. Edward Ardern. (Paper read before the Assoc. of Managers of Sewage Disposal Works.) (104) Dec. 18.
Emergency Garbage Disposal in Chicago; Acid Treatment in Vats and Burial in Clayholes under Successive Layers of Ashes Proves to be a Successful Temporary Measure.* George B. Young. (14) Dec. 19.
Tests Controlling Sewage Plant Operation. (14) Dec. 19.
Unwatering Carson Lake.* L. D. Davenport. (16) Dec. 19.
The Manufacture of Concrete Sewer Pipe. (101) Dec. 25.
Sewers Cleaned with Scrapers to Pass Ball Test; Methods and Costs of Work, Involving Pipe from 8 to 18 Inches in Diameter, at Carlisle, Pa.* Clark A. Bryan. (14) Dec. 26.
Reducing the Cost of Drainage Excavation. (14) Dec. 26.
Report of N. C. G. A. Committee on Heating, Ventilation and Refrigeration. George S. Barrows. (24) Dec. 28.
An Exhibit of Modern Street-Cleaning Apparatus: Refuse Collection.* (13) Dec. 31.
Preliminary Report on Emscher Tanks and Kindred Sewage-Clarification Processes. K. Thumm and C. Reichle. (Abstract from Communications of the Royal National Inst. for Water Hygiene in Berlin-Dahlen.) (13) Dec. 31.
A Snow Removal Plan for New York City. (Abstract of Report by J. T. Fetherston.) (13) Dec. 31.
Heating Equipment for Large Greenhouse.* George W. Loeber. (101) Jan. 1.
Zweckmässige Gestaltung der Fensternischen, in denen Heizkörper Aufstellung finden. H. Chr. Nussbaum. (7) Oct. 10.
Die neuen Sielbauten der Stadt Altona.* von Mouillard. (80) Oct. 22.
Die Warmwasserheizungs-und Warmwasserbereitungsanlage in der Privatklinik des Sanitätsrates Dr. med. E. Lampé in Frankfurt a. M.* (7) Oct. 24.
Die Sanierungsarbeiten der Ueberschwemmungszone des brasilianischen Staates Rio de Janeiro.* Friedr. Freise. (7) Oct. 24.
Untersuchungen über elektrische Heizung in Stockholm.* Carl A. Rossander. (41) Dec. 3.

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Structural.

Reactions in a Three-Legged Stiff Frame with Hinged Column Bases. N. M. Stineman. (4) Nov.
Building Laws. Henry Rusch. (Paper read before the Engrs'. Club of St. Louis.) (1) Nov.
Permeability Tests on Gravel Concrete.* Morton O. Withey. (4) Nov.
Aluminum Alloys as Surface Protection for Metals Subject to High Temperatures. (13) Dec. 3.
Sulphite Solution Plant of Reinforced Concrete; Four 99-Foot Towers, 8.5 Feet in Diameter with a Special Tile Lining, Contain the Limestone.* (14) Dec. 5.
Dustless Concrete Floors.* (14) Dec. 5.
Erecting 36-Ton Girders with Steel Shears.* (14) Dec. 5.
Measuring Shocks of Vehicles and Vibrations of Buildings.* A. Boyer-Guillon. (From *La Science et la Vie.*) (19) Dec. 5.
Acoustics of Auditoriums.* F. R. Watson. (Abstract from *Bulletin 73*, Eng. Experiment Station, Univ. of Illinois.) (19) Serial beginning Dec. 5.
The Care and Maintenance of Gas Holders. J. H. Braine. (Paper read before the Am. Gas Inst.) (24) Serial beginning Dec. 7.
Using Water as Load in a Floor Test. F. R. McMillan. (13) Dec. 10.
The Palmer Memorial Stadium at Princeton University.* (13) Dec. 10.
Favorable Tests of Elmira Brick. (14) Dec. 12.
Behaviour of Reinforced Concrete in Fires; in Three Instances Buildings were not Injured by Burning of Inflammable Contents or Destruction of Adjoining Structures. (14) Dec. 12.
Cost of Floors for Buildings. (14) Dec. 12.
The Design of Railroad Ice Storage Houses. (Report of Committee of the Am. Ry. Bridge and Building Assoc.) (86) Dec. 16.
A New Flat-Slab Reinforced Concrete Floor with Striking Features.* (13) Dec. 17.
The Fire at the Edison Works at West Orange, N. J.* (13) Dec. 17.
Rapid Destruction of Fireproof Buildings (Edison Works).* Sterling H. Bunnell. (20) Dec. 17.
Large Floating Gravel-Washing Plant in Indiana.* (14) Dec. 19.
Disastrous Fire at Edison Factory.* (14) Dec. 19.
Painting Defects, Their Causes and Prevention.* G. W. Thompson. (Paper read before the Maintenance of Way Master Painters' Convention.) (15) Dec. 18.
Erecting 40-Ton Girders at a Height of 250 Feet Above the Street.* (14) Dec. 26.
Manila Rope Fastenings; Tests of Tensile Strength of Common Hitches Indicate That Eye-Splices are Most Satisfactory Considering High Strength They Develop.* (14) Dec. 26.
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An Example of Fire-Resistant Windows and Structure.* (13) Dec. 31.
The Corrosion of Iron. L. C. Wilson. (9) Serial beginning Jan.
Ermittelung der Abmessungen einfach und doppelt bewehrter Eisenbeton-Querschnitte bei reiner Biegung sowie exzentrischem Druck und Zug. Stark und Dankelmann. (51) Sup. No. 22.
Der Neubau der Berliner Sternwarte auf dem Babelsberg.* W. Eggert. (49) Pt. 10.
Pfahlgründung mit Holzeisenbetonpfählen.* Neufeldt. (40) Aug. 12.
Die Grosse Maschinenhalle auf der Buchgewerbe-und Graphik-Ausstellung in Leipzig.* Kottke. (69) Sept.
Der Knickwiderstand Gegliederter Eisenstäbe.* Siegmund Schwätzer.* (69) Sept.
Das Natürliche Recht des Eisens auf Aesthetisierung.* Karl Stapf. (69) Serial beginning Oct.
Die Halle für den Kessel- und Gestellbau des neuen Transformatoren-Werkes der Siemens-Schuckertwerke in Nürnberg.* M. Foerster. (69) Oct.
Einfluss des elektrischen Stromes auf Beton. (80) Oct. 24.
Fundamentplatte in geteilter Eisenbetonkonstruktion.* H. Martin. (78) Nov. 6.
Mehlmagazin mit Backgebäude.* Ernst Schick. (78) Nov. 6.
Eine Studie über Zugspannungen. H. Wald. (50) Nov. 12.

Topographical.

Cross-Sectioning.* J. A. Macdonald. (96) Nov. 26.

Water Supply.

Construction Features of the North Side Reservoir of the Water Works of the City of Pittsburgh.* E. E. Lanpher and J. S. Cole. (58) Oct.
Ice Troubles in Hydro-Electric Plants. (96) Nov. 26.
Water Supply Main for Atlantic City, N. J.* (60) Dec.
The Hydro-Electric Power Plants at the Wachusett Dam, Clinton, Mass.* B. C. Thayer and E. R. B. Allardice. (109) Dec.
Twelve Miles of Concrete Pipe Laying, Eastern Colorado Power Co. (67) Dec.

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First Large American-Built Humphrey Pump.* Charles C. Trump. (64) Dec. 1.
British Practice in the Design of Reinforced Concrete Reservoirs. E. R. Matthews. (Paper read before the Univ. College, London.) (86) Dec. 2.
A Small Slow Sand Water-Filter for the Estate of J. P. Morgan.* John H. Gregory. (13) Dec. 3.
A New Hydraulic Stop Valve.* C. W. Larner. (13) Dec. 3.
Public Water Supply for Cities—Some General Considerations. W. H. Dittoe. (96) Dec. 3.
Stream Flow Investigations.* M. C. Hendry. (96) Dec. 3.
Administration of Water Rights in British Columbia. William Young. (96) Dec. 3.
Economical Duty of Pumps.* Frank H. Carter, Assoc. M. Am. Soc. C. E. (14) Dec. 5.
Bacteriological Standard for Water on Common Carriers. (14) Dec. 5; (13) Dec. 17.
Solving the Silt Problem; Reservoirs Suggested for Muddy Streams of the Southwest. Louis C. Hill. (14) Dec. 5.
Method of Lining Wrought Steel and Cast Iron Pipe with Cement.* (86) Dec. 9.
Storage Dam at Eugenia Falls.* G. R. Heckle. (96) Dec. 10.
The Senfrot Chemical Injector.* (11) Dec. 11.
Thermophones in Kensico Dam.* Wilson Fitch Smith. (13) Dec. 10.
Water Sedimentation, Coagulation and Mechanical Filtration at Waco, Texas.* (13) Dec. 10.
One Year's Work of the U. S. Reclamation Service. (14) Dec. 12.
Grouting Foundations for Gravity-Section Dam.* Harold S. Johnston. (14) Dec. 12.
Discharge of Water Through Nozzles; Derivation of Formula for Use with Slide Rule and Adapted to Introduction of any Efficiency Factor Desired. F. W. L. Peebles. (14) Dec. 12.
Some Observations on Water Works Valuation. J. W. Ledoux. (Paper read before the Pennsylvania Water Works Assoc.) (86) Dec. 16.
Bow River Hydro-Electric Plants.* (96) Dec. 17.
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Plant Tests of a Low-Head Hydro-Electric Development.* F. Nagler. (13) Dec. 17.
A Direct-Reading Volumetric and Velocity Meter for Measuring Fluid Flow.* (13) Dec. 17.
The Narrows Flexible Joint Submarine Siphon.* John P. Hogan. (14) Dec. 19.
Reservoir Sites on the Colorado River.* Louis C. Hill. (14) Dec. 19.
Winter Operation of Stoney Gates.* (14) Dec. 26.
Some Recent Tests of High-Power, High-Speed Water Turbines; Supplementary Article Dealing with Holyoke Test 2 359, Giving 93 Per Cent. Maximum Efficiency, and 2 363, Giving 90.7 Per Cent.* S. J. Zowski. (14) Dec. 26.
Cost of Excavating Drainage Ditches with Steam and Electric Machines; Comparisons of Drag-Line and Suction Dredges Operated on the North Side Minidoka Project of the United States Reclamation Service. (14) Dec. 26.
A Diesel Engine Irrigation Plant.* Max Loewenthal. (111) Dec. 26.
A Discussion of the Elements of Water Works Accounting.* R. A. Stevenson. (86) Dec. 30.
The Protection of Riveted Steel Pipe. Leonard Metcalf. (86) Dec. 30.
Design Features of the Proposed Water Supply and Purification Works, Corpus Christi, Texas.* (86) Dec. 30.
The Influence of Reservoir Bottoms on Stored Water.* Wilson Fitch Smith. (13) Dec. 31.
Outline of Data to be Obtained for a Report on the Feasibility of any Irrigation Project. H. F. Robinson. (13) Dec. 31.
Construction Camp, Elephant Butte, N. M.* (13) Dec. 31.
Irrigation Flumes of Ingot Iron.* (13) Dec. 31.
Umbau einer offenen in eine geschlossene Enteisenungsanlage und dabei gesammelte Erfahrungen. Krecke. (39) Serial beginning Oct. 5.
Der Gebrauch des Chlorkalks zur Trinkwassersterilisation.* E. A. Gieseler. (7) Oct. 24.

Waterways.

Long-Term Variations in Stream Flow, Croton and Hudson Rivers.* Edward H. Sargent. (13) Dec. 3.
The Floating Caisson for the Panama Canal Locks.* Lewis A. Mason. (13) Dec. 3.
Flood-Protection Work on Fall Creek at Indianapolis, Ind.* L. F. Wertz. (13) Dec. 3.
Lock Gates for Dalles-Celilo Canal.* (14) Dec. 5.

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Tidewater Activities at Halifax.* **(96)** Dec. 10.
Flow Regulation on the Grand River. **(96)** Dec. 10.
The Gulf Stream, Causes that Govern its Course, and its Effect on Climate.* M. W. Campbell Hepworth. (Paper read before the Royal Geographical Soc.) **(19)** Dec. 12.
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PAPERS AND DISCUSSIONS

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RIVERS AND RAILROADS IN THE UNITED STATES.

By William W. Harts, M. Am. Soc. C. E.

To be Presented February 3d, 1915.

Synopsis.

The large appropriations of public funds for the development of the channels in our rivers and harbors have grown from very modest beginnings.

Before the application of steam to transportation, the principal methods of communication were by horse-drawn vehicles or by sails. The invention of the light-draft river steamboat offered many facilities for the development of large areas otherwise inaccessible, and the demand for better channels in our interior rivers at once grew. Later, some of our harbors required better facilities, so that, from that day to the present, expenditures for these purposes have increased year by year until now there are projects under construction, or favorably considered, amounting to more than $289 000 000, and the annual appropriations have increased to more than $40 000 000.

On our rivers, the engineering methods adopted for accomplishing the work for which projects have been prepared involve all the best principles of river engineering in the world. They are surpassed nowhere in their excellence of design, suitability to the work for which

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proposed, economy of construction, and success in operation. On the economic side, however, there are new questions to be solved involving returns on the outlay. It is in this section of the subject that surprises are met, and many disappointments beset the river engineer.

In spite of the enormous sums of money expended on the rivers of the Mississippi Valley, there are but few that are holding their own as commerce carriers. The great majority are dwindling in traffic, so that it is apparent that their present condition, as to the depth and availability of their channels, is in advance of the use being made of them. Whenever streams do not show an increasing traffic, which is at least approaching the increase of commerce in the region traversed, it shows that the usefulness of the stream in question is declining, and the necessity for continuing large expenditures is open to doubt.

There seem to be three fairly distinct stages in river history; the first covers the period commencing with the earliest times, the application of steam to vessels, and continuing up to the expansion of railroad building; the second stage covers the period from the rapid expansion of the railroads up to recent years; the third, in most sections of the United States, is still in the future. During the first period, commerce on the streams increased rapidly with the development of industrial conditions. During the second period, the railroads have taken over most of the river business, leaving the rivers struggling for a minor share of the traffic. During the third period, the prosperity and density of the population of the vicinity will afford sufficient traffic for railroads and rivers together.

In general, the United States, principally the central part, is now in the second stage, and the public is beginning to realize the necessity of curtailing expenditures until the time shall come when the industrial conditions will prove beyond question the need for using our rivers for special kinds of freight.

This disappointing record of interior streams does not apply to most of our harbors, and many streams on the coast are exceptions to the general conclusions mentioned; but there can be no doubt that, in the interior development of the nation, the usefulness of interior rivers as lines of communication has become much less important in late years. There are growing evidences that this is being perceived by the public, and we may soon look for restrictions in the large expendi-

tures now being made for all projects not showing a healthy and profitable growth in traffic.

In most of the progressive countries of the world, especially in those where population is dense and production large, the improvement of the waterways by the Government, for better communication in the interior, and the protection and deepening of harbors, have gone on at a rate which, as a general rule, has kept pace with the increase in population and production. Of late years, however, this progress has been strongly affected, both in America and in Europe, by the development of railways—sometimes favorably and sometimes otherwise.

In the United States the improvement of rivers by Federal appropriations extends far back into the early history of the nation, long before the era of railways. Before the first locomotive ran on rails and before the first steamboat had demonstrated its practicability, communication between different parts of the country was maintained chiefly by the sea, the interior being wholly dependent on roads and canals, and horse-drawn vehicles. Even during this time, while the interior was undeveloped and the population restricted to a comparatively narrow belt along the seacoast, these means were inadequate, and in proportion as the early pioneers conquered the unopened regions to the westward, the necessity for better communications became more and more urgent.

The Federal Government in those early days was not so compactly knit in its administrative processes as at present, and the improvement of interior waterways by Federal assistance was not admitted generally to be a proper Governmental function. These matters were largely left at that time to the initiative of the individual States; but the result of this policy was soon found to be so unsatisfactory and the road and canal work of the country so badly co-ordinated, that in 1824 all this improvement was placed under the charge of Army Engineers and given Federal support, in the hope that the early deficiencies might be met and obviated.

Work on these canals and highways required many engineers. Up to 1824 the Military Academy at West Point was the only school in the country which trained men as engineers, and for many years thereafter it was the principal school of this kind. It was only

natural, therefore, that the Government should have looked to its specially trained men to lay out and construct the work that was being done by the country at that time.

With the coming of the steamboat, the Government's interest in canal and highway work was soon expanded to include the improvement of rivers and the deepening and protection of harbors as the necessity therefor grew, so that, by a natural process of development, the duty of designing and constructing the enormous waterway projects which have since come into existence was the logical inheritance of the Army Engineer.

Until 1860 this work was confined mainly to the improvement of interior rivers, harbor work being still comparatively unimportant, although the project for the mouth of the Mississippi River, in all essential respects similar to what was constructed by the late J. B. Eads, F. Am. Soc. C. E., in 1874, was designed and proposed by an Army Engineer Board consisting of Colonels Barnard, Beauregard, and Howell, in 1854.

The light river steamboat jumped into favor at once as a popular method of communication, and showed very early that some improvement of the interior streams was plainly necessary. Many river cities, such as St. Louis, Memphis, and Cincinnati, trace their original importance as commercial centers to their favorable location on navigable streams. Up to this time, the increase in depth of ocean-going vessels had not had a very strong influence on the deepening of our harbors, for many ports having adequate channels could be found along our coast. Our rivers, therefore, received the earliest attention, and established first their claims to Federal aid. This work increased in volume and effectiveness up to the time of the Civil War, but all such work was abandoned during that conflict. In 1867, river improvement was again resumed, and the impetus received by all commercial business after the war was felt in this kind of work as well. At present the river and harbor projects, so modest in the beginning, have grown so that the work requires more than $40 000 000 annually.

The very early records of public money expended are so incomplete that they can scarcely be used to illustrate the enormous growth of these works, but between 1875, a comparatively recent date, and 1914, the annual appropriations increased from $5 218 000, in round numbers, in 1875, to $53 000 000 in the proposed Act of 1914, or more than

eightfold in about 40 years. These large sums are appropriated and expended in accordance with projects prepared by the Army Engineers and approved by Congress. The responsibility, therefore, must be shared between these two in some proportion, however unequal.

Very few people realize the amount of river and harbor improvement already adopted or under construction. Projects which have already received the sanction of Congress and are under process of construction now amount to more than $200 000 000. About $86 500 000 in new work has been reported favorably by the Engineer Department, but has not yet been adopted by Congress. In addition, there are projects amounting to more than $2 500 000, which have been favorably considered, but not yet accepted by the Engineer Department. Thus, there is a total of work costing more than $289 000 000, which is either in progress, or approved by Congress, or soon to be brought before the country.

Among the prominent works under construction, there are now 61 projects on the Great Lakes, the total estimated cost of which is more than $63 000 000; on the Atlantic Coast, 200 projects involving more than $100 000 000; on the Gulf Coast, 42 projects involving nearly $20 000 000; and on the Pacific Coast, including Alaska and Hawaii, 28 works requiring $22 000 000. The total of more than 300 different projects requires an expenditure of more than $205 000 000. It may be interesting to know that the work done on these projects up to June 30th, 1913, on the Great Lakes, has been carried on at an actual cost of 3.9% less than the estimates; on the Atlantic Coast, the actual cost has been 11.1% below the estimates; on the Gulf Coast, 8.7% below the estimates, and on the Pacific Coast, 8.8% above the estimates, the total for work done, up to June 30th, 1913, being 6.17% below the estimates.

It has been officially reported that in the 100 years, from 1802 up to December, 1902, there had been spent by the United States Government $221 869 759 for rivers, $147 448 903 for harbors, and $33 237 857 for canals, or a total of more than $400 000 000. From 1814 to 1900, France spent $449 000 000 on new works and in the maintenance of those already built. Similarly, Belgium spent $101 000 000 between 1831 and 1903. Between 1813 and 1906, Prussia spent $129 000 000 for new construction alone, and her maintenance charge in the single year, 1905, was $4 000 000. The United States is

about eighteen times as large in area as either France or Germany, and has about four times the length of navigable streams. Although the density of population in the United States is much less, being only about one-half that of all Europe, and about one-twelfth that of Germany, it can be seen that the United States has more than kept pace with other progressive nations, when density of population is considered.

As to the character of the work done by the United States in the improvement of waterways, reference may be made to the report of the National Waterways Commission of Congress sent abroad a few years ago to study waterways in Europe. In this report, it is stated that America had but little to learn from European works. This referred mainly to the excellence of the engineering features of our own waterways. A study of the methods and appliances in use in older countries convinced the Commission that there is at present no modern successful plan for the improvement of waterways which has not been better done in America. Furthermore, the engineering difficulties are of greater variety and magnitude and of more serious character here than in any country abroad.

To illustrate the high character of the work already accomplished, the Upper Mississippi may be pointed out as a model of open river regulation which, for this kind of improvement, is excelled nowhere else in the world. The canalization projects of the Great Kanawha and the Ohio, using movable dams, are models of their kind. The training of the mouth of the Mississippi River is an excellent and widely known example of the successful improvement of river mouths in tideless seas. The deepening of the harbor entrance at Galveston by jetties, the improvement of the entrance to the harbor of New York by deep dredging, and the construction of locks in St. Marys River, are all noteworthy examples of successful and highly developed engineering works which stand pre-eminent in their class. Even the system of storage reservoirs at the head of the Mississippi River is the largest and best managed of any in the world, and the system of levees on the Lower Mississippi, although not complete, has developed a standard and pointed the way for all future work of this kind. The country may thus take pride in the accomplishments of its engineers, and give deserved credit to them for initiative, courage, and skill. Excellent

engineering facilities for an enormous water-borne commerce have already been provided for, and are still being added to.

The first question that will be asked the river engineer is, to what extent are these facilities used, and do they pay in public service for the enormous cost of construction and maintenance? When we begin to study the economic features connected with works of this character, we are struck with surprises. The advent of the light-draft, inexpensive steamboat made our rivers and waterways of great value as commercial carriers at a very early date. These vessels, built with great propelling power and long, balanced rudders, carried enormous loads on very light displacements, often transporting several trainloads capacity of that period on hulls having a draft of less than 6 ft. The shallow and often swift rivers were responsible for this type, which was evolved after bitter experience. So successful did these vessels become soon after the Civil War that there were but few rivers in the Mississippi Valley that did not boast of their regular packets, often boats of great elaborateness of construction, giving rise on the Mississippi to the term "floating palaces". Too much praise cannot be given to this simple means of developing a great region; and, indeed, it has been asserted that the navigation on the Mississippi River had a direct influence on the result of the Civil War, as the control of its outlet by a separate government was viewed with great alarm by the residents of the upper valley who saw the possible adverse influence on their future development, and understood more clearly from this concrete argument their necessity for a continuance of the Union. The hazards of this sort of transportation, however, were very great, and the steam railroad was beginning to be a fierce competitor. Railroads found a fertile field in those areas where the rivers had already induced a new prosperity. As these railroads were developed throughout the country into that intricate network which is the pride of the nation to-day, little by little, the freight formerly handled by river was taken over by the railway lines, until now it is found that practically all the rivers of the great Mississippi Valley are dwindling in commerce in spite of the large expenditures for better channels that have been and are still being made. This falling off in usefulness is in the face of an enormous expansion in production of all kinds and a development of commerce that is unsurpassed in our history.

To illustrate, compare the commerce of the Kentucky River in 1892, amounting to 431 846 tons, with that of 1912 of 186 300 tons as shown by the Reports of the Chief of Engineers—a loss of nearly 60% in 20 years. In 1892, there were but five locks in the lower river, built by the State of Kentucky, permitting the use by vessels of depth from the Ohio to above Frankfort, 65 miles, whereas, in 1912 there were twelve locks furnishing a 6-ft. channel to a point 239.5 miles above the mouth. Thus it is found that, notwithstanding a great increase of facilities, there has been a decided reduction in commerce on this river. This condition was foretold by the engineer officer in charge of this work in 1895, but the progress of work could not be stopped, owing to the insistent demand of the locality for further improvement.

The Ohio River, which has held its commerce better than most of the interior streams on account of the coal deposits at its headwaters, has diminished in commerce from a total of 13 000 000 tons in 1905 to 8 618 000 tons in 1912, approximately 35% in 6 years. Between 80 and 90% of this business is in coal, which is carried on the river mostly at high stages, when works of improvement are not so necessary as at lower levels. The commerce of the Mississippi River at St. Louis has dwindled from 1 208 205 tons in 1892 to 265 720 tons in 1912, a loss of nearly 80% in 20 years. The commerce of the Green River in Kentucky has diminished from 462 208 tons in 1892 to 306 910 tons in 1912. On all the streams of the Mississippi Valley, there are few on which the commerce is not diminishing. The Great Kanawha barely holds its own in tonnage; the Cumberland River shows about 10% increase in traffic in 20 years; and on the Tennessee River there was also a considerable increase up to a few years ago, but a diminution since then. Almost all the others are losing their traffic. Such a record is indeed a disappointment to all those who were interested in fostering river commerce.

The Mississippi Valley is one of the richest and most productive areas in the world. The volume of commerce originating there is enormous, and has been rapidly increasing during the past 15 or 20 years, but, notwithstanding this growth, less and less is being carried by the rivers, and only those few streams which are not paralleled by railroads are still managing, with many a struggle, to maintain their former value to the public. Nowhere are the products of the farm more valuable; nowhere are the mines more productive; and nowhere

are the energy and capacity of its people excelled in all those various pursuits of wealth which are so numerous in this great area. If, therefore, the rivers anywhere might be expected to show increased usefulness as the country develops, it would certainly be in this region. It is, therefore, with keen disappointment that the opposite tendency is found to be growing more than ever apparent. Rivers in this valley were teeming with steamboats and barges within the memory of men still living. The arrival and departure of the large packets were events of much interest in the various towns where steamboat landings were scheduled, and boating was a well recognized industry, employing many men. To-day, however, the passenger traffic by river boat has almost disappeared, and bulky and slow freight forms the main part of what is left of a once flourishing business.

The explanation of this is not far to seek. The railway of to-day is well able to compete with the waterways at every turn. Combinations of many small and weak lines into through routes, and the extension of rail lines into every region where it seemed reasonable to expect a financial return, have developed what was a disconnected and feeble collection of roads into a systematized network of enormous value. The unfeeling and unrelenting competition which commenced first between river and rail lines and then continued among the various rail lines themselves, has forced an economy of operation and administration which has made the rail lines a giant in power and a miracle of usefulness. Within little more than a generation, the average cost of moving a ton of freight a mile in the United States has diminished from 7½ cents to about 7½ mills, a reduction of nine-tenths, and some of the coal roads having easy grades and flat curves boast that their tonnage cost has been cut to 2.3 mills per ton-mile. This is the harsh competition that rivers must meet. Railroads can be changed in location, and terminals can be placed wherever needed; cars can be switched from one line to another; spurs can lead to the point of destination without transhipment or breaking bulk, and distribution is simplified. On the other hand, waterways are fixed in location, require numerous expensive terminals, need constant improvement, and require steamboats and other craft of considerable cost. River lines have the advantage that their roadbed is provided by the Government. Railroads require large sums for general administration; for employees in yards and at stations; for maintenance of way and

equipment; for interest on their capital invested, and taxes. As the steamboat has no maintenance-of-way expense, it has a great advantage over the railroad in regard to fixed charges. Its interest on its cost has been estimated at 5%, insurance 8½%, and maintenance at 7½%, whereas the railroads must pay interest on the cost of the entire road, estimated at 5%, maintenance estimated at 2%, interest on the cost of equipment 5%, maintenance of equipment 10%, and insurance 3 per cent. Notwithstanding this unequal burden, railroads have out-distanced the river in economy of administration, and on most roads the fixed charges for interest on capital and maintenance are much less per ton-mile than those on a river packet. This indicates clearly the reason for the decline of the river commerce of St. Louis in 30 years from 2 120 825 tons in 1880 to 191 965 tons in 1910, a loss of about nine-tenths. During the 17-year period, from 1890 to 1906, the river commerce of that city dwindled from about 1 260 000 tons to about 317 000 tons, but in the same period the rail business increased from 15 000 000 to about 45 000 000 tons, or about 300%, according to the reports of the St. Louis Merchants Exchange. As a competitor, the Mississippi River has fallen from a position of pre-eminence to almost a negligible quantity.

Furthermore, of late years, a great decrease in the cost of handling freight on railroads has taken place, notwithstanding increases in cost of materials, labor, and taxes. On the other hand, an increase of at least 50% in steamboat operation costs has taken place, due to these same advances, but this has not been offset by reduced operating expenses. The river steamboat has not changed much in late years, and river terminals have nowhere been improved to a marked degree; certainly these improvements have not kept pace with the improvement of the channels by the Government. It has been stated that the stock in the largest company plying between Louisville and Cincinnati has fallen from a high premium to less than par within the last 25 years, a loss of more than 80 per cent.

Thus, it is seen that railways are driving steamboat commerce from most of our interior streams. This was started years ago by vicious hostility and unfair competition on the part of the railroads, but now the control by legal means of these unfair practices has left only general economic laws in operation, which, after all, form the safest basis for every industry of this kind.

The realization of the one-sidedness of this unequal struggle has of late years brought with it to river engineers a feeling that we are now far ahead of the demands of the present in many of our inland streams, and henceforth expenditures should be restricted to the barest necessities until the commercial development of the neighboring regions brings with it new demands. It seems not too strong a general statement to make that the navigational facilities on every stream on which commerce is now diminishing is far in advance of the present necessities, and that additional improvement at public expense should be withheld until such time as the economic pressure for additional transportation facilities becomes plainly manifest.

It seems reasonable to suppose that these tendencies toward diminution of river commerce are not necessarily permanent everywhere, for the commercial development of the areas contiguous to transportation lines is nearly always conspicuous, and frequently brings a need for additional facilities. New railroads are not being built so rapidly as a few years ago, and some day some of the interior streams may be expected to handle an increasing commerce in bulky freight where time of transit is not of great importance. This need for meeting the demands of expanding production may bring back some rivers to a new usefulness that cannot now be safely predicted.

Nor must we conclude that all our rivers are losing their value as commerce carriers, or that all are even diminishing in usefulness. Many streams outside the great central valley of the country, particularly those that empty in the harbors along the coast, and allow ocean-going vessels or coastwise ships to reach interior points, are showing very encouraging results. Some of those entering New York Harbor, for example, in the regions where population is dense, are very valuable and carry large quantities of commerce. In 1905 Arthur Kill had a commerce of 11 700 000 tons, valued at $265 000 000; and in 1911 it had a commerce of 30 500 000 tons, valued at $515 400 000.

In looking over the history of some of our rivers, it appears that they pass through several more or less indistinct stages of usefulness. First, while the country is comparatively undeveloped, and before the construction of railways has been begun, the rivers are found to be the best and cheapest lines of commerce. At such times the steamboat enterprise thrives and river commerce multiplies. In some cases

new land is opened to profitable cultivation, or new industries are encouraged. Later, towns spring up, and a whole region often receives a new impetus from its greater accessibility. Next comes the period when the results of this enterprise have brought about such a prosperous condition that the extension of railways is induced in the same territory. They, with their many advantages, then absorb most of the river transportation business. River commerce dwindles during this period, even though increase in production is noticeable on every hand. At such time discouragement may be felt by river advocates, because it seems that the rivers are not performing their wonted part in the upbuilding of the country through which they pass, for the part played by the streams is lost sight of in the general economic progress of the neighborhood.

This discouragement might be justified but for the third stage, which seems to be within the range of safe prediction for some of our internal waterways, especially if one judges by the rivers emptying into New York Harbor and those rivers like the Rhine flowing in congested communities. This third stage comes when the population and production have increased to such an extent that all lines of traffic are insured a large part in the transportation business of the locality. At such times the rivers again become useful and efficient. If we have proceeded too rapidly in the improvement of our interior rivers, there is still this hope of ultimate usefulness after the second stage has passed.

On the other hand, however, our harbors have had no such vicissitudes. Economy of ocean transportation has gradually forced shipowners to adopt deeper draft for their vessels. With the advent of steam, the depth of hull has steadily increased until now the largest ships are limited to few ports. The harbor having a depth of 25 ft., which was ample a few years ago, is now no longer sought by the large ocean vessels, unless the channels have been deepened to meet the new requirements. The entrance to New York Harbor, which had been ample for many years, has recently been dredged to a depth of 40 ft. to provide for this pronounced increase. Norfolk, Philadelphia, Savannah, Galveston, and many other harbors have also been compelled to provide for the increased draft of vessels.

The expansion of the railways throughout the interior, in many cases, has led directly to the greater development of the seaports.

Those who remember the almost feverish haste in building railways toward Galveston, when the harbor depths were first made ample for sea-going vessels, will see in that movement the importance of deep water when near productive areas.

The harbor, by its very nature, is a sort of terminal where products are exchanged between rail and water. Thus the railway, which is now having such an adverse effect on some of our interior rivers, has at the same time brought about a corresponding necessity for improvement in many of our harbors, and has contributed enormously to their value.

For example, in 1892, in New York Harbor, the exports and imports of foreign trade alone amounted to about 5 000 000 tons. In 1912, it totaled more than 14 000 000 tons, an increase of nearly 300 per cent. The total commerce by water was estimated in the 1906 census report at 114 000 000 tons. The harbor at Norfolk in 1892 had a commerce of 3 427 000 tons, whereas in 1912 it was more than 22 000 000 tons, an increase of nearly sixfold. Savannah in 1892 had about 2 000 000 tons of commerce, and in 1912 more than 3 120 000 tons, an increase of more than 50 per cent. Galveston in 1892 had a tonnage of 1 134 326 tons and in 1912 a total of 3 224 367 tons. These cases are probably the more conspicuous ones, but it may be accepted as a general rule that all the larger harbors have amply justified the expenditures made in providing better channels.

It is a wise policy, amply justified by experience, to have harbor facilities always a little in advance of the immediate necessities, and many of our seaports have responded admirably to efforts in this direction. It seems not unreasonable, however, to suggest that the liberal treatment which our interior streams have received could be restricted very materially, in the interest of economy.

On the whole, our expenditures for waterways have been of immeasurable benefit to the country at large. Never in our history has the volume of domestic and foreign commerce been so great, and never has the outlook for future increases been brighter. Our over-sea exports and imports are now growing enormously, and the prosperity of the land requires that there should be no restriction anywhere on account of inadequate channels. A practical test, almost infallible in its application, that will show whether a waterway project can be economically considered for further improvement, is a progressively

increasing commerce, and the measure of saving in cost of transportation will always be a guide as to the extent of work that is justified.

Thoughtful observers of our system of channel development have been impressed with the tendency of the Government to over-liberality in many instances, but the signs are multiplying that, in the near future, the public will demand a more rigid adherence to economic laws in the adoption of new projects.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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RECONSTRUCTION OF THE NORFOLK AND WESTERN RAILWAY COMPANY'S BRIDGE OVER THE OHIO RIVER AT KENOVA, WEST VIRGINIA.

By William G. Grove,* Esq., and Henry Taylor,
Assoc. M. Am. Soc. C. E.

To be Presented February 17th, 1915.

Synopsis.

The purpose of this paper is to set forth in a general way, by using a specific case, the difficulties that engineers must surmount when it becomes necessary to replace an existing structure, which has become inadequate for the demands of modern service, with another structure capable of meeting present and future conditions.

In this instance, the problem consisted of replacing a single-track, five-span bridge, 1 800 ft. long and 100 ft. above water level, with a double-track bridge, on the same piers, without interruption either to railway traffic or river navigation.

The methods used in the reconstruction are described as well as the design of the new structure. The paper is divided into eight parts, as follows:

History and description of the old bridge;

Erection of the viaduct approach;

Various preliminary schemes for the erection of the river spans;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Asst. Engr., Eng. Dept., American Bridge Co., New York City.

Description of the new spans, with loads and unit stresses used in the design;

Method of remodeling the tops of the present piers;

Description of the pier girders and material tracks;

Method finally adopted and used for the reconstruction of the five river spans;

Arrangement of the material yard.

The writers, as far as possible, have endeavored to make the paper a descriptive one, and hence have omitted all the calculations necessary in the design and in ascertaining the deflections of the several spans. The calculated figures, however, agreed very closely with the distances actually measured in the field.

By the addition of drawings and photographs, it is hoped to show the actual field conditions in a much clearer way than could be done by description. Figs. 46 and 47, showing the flood of March, 1913, tell better than words why the extra precautions were taken during this reconstruction.

By this method of description and illustration, the writers hope to make the paper interesting and instructive to those members of the Society who are not connected with the structural end of engineering, as well as to bridge engineers.

In conclusion, the point to be emphasized is the practicability, from an economic as well as from an engineering standpoint, of safely reconstructing existing structures without serious interruption of traffic.

Introduction.

The original single-track Norfolk and Western Railway bridge across the Ohio at Kenova, W. Va., having become inadequate for the heavy traffic of the road, it was decided in 1909 to replace it with a heavier double-track structure. As this bridge was on the only practical route for the Railway Company's traffic, the problem presented to bridge builders was one of replacing the bridge and at the same time not disturbing the traffic over it, or river navigation. As the tops of the piers were about 100 ft. above the water, and as the construction of new piers would have involved quite a considerable additional expense, the question ultimately resolved

itself into one of replacing the bridge with a new structure on the same piers.

The paper describes the manner in which this reconstruction, by the method of cantilever erection, was accomplished successfully by the American Bridge Company during 1912-1913, without any interference with river navigation and, except for one day, without interruption of railway traffic.

History and Description.

The Town of Kenova is on the south bank of the Ohio River, in West Virginia. It is just opposite the extreme southern part of Ohio, and is separated from that State by the Ohio River, and from Kentucky by the Big Sandy River which empties into the Ohio at this point, hence the derivation of the name: Ken for Kentucky, o for Ohio, and va for Virginia, and it is therefore called the Tri-State City.

At this point the Ohio River flows from east to west. On the north or Ohio shore the bank is very steep and the hills extend almost to the river bank. On the south shore, however, conditions are different. The hills begin to rise about ½ mile back from the river, and Kenova is on the plain between the hills and the river. The tracks of the Baltimore and Ohio Railroad and the Chesapeake and Ohio Railway run parallel to the Ohio River on the south bank. The Norfolk and Western Railway comes directly out of the hills on the south bank and, after passing along a viaduct over the Chesapeake and Ohio and the Baltimore and Ohio lines, crosses the Ohio River at right angles and at an elevation of about 100 ft. above mean water level. Trains cross the bridge at the rate of 72 per day, or about 1 every 20 min., and consist of from 50 to 75 cars each. These trains are made up principally of coal cars, bringing coal from the fields in the mountains of West Virginia and transporting it through Ohio, Indiana, etc.

The old bridge, Fig. 1, completed in 1891, was fabricated by the Edge Moor Bridge Company (now the Edge Moor Plant of the American Bridge Company), and was erected by Messrs. Baird Brothers. It consisted of five single-track, through, pin-connected truss spans, a channel span of 518 ft., flanked on each side by two 298-ft. spans. There was a 2 148-ft., single-track, deck, plate-girder, viaduct approach on the West Virginia side and one 64-ft., single-track, deck, plate-girder span on the Ohio side. The old bridge was designed for

Cooper's E-40 live load. The trusses were made 34 ft. from center to center, with the idea of putting a third truss in the center when it became necessary to double-track the bridge. All the spans were of the sub-panel type. The bottom chord was made up of two lines of eye-bars, 7 ft. 0 in. apart vertically; in the end panels, however, the two lines were brought together so as to connect to the single pin at the shoe. Floor-beams and stringers were placed between the two lines of eye-bars. In the intermediate panels, there were two sets of bottom laterals, one in the plane of the top flange of the floor-beams and the other in that of the bottom flange. In the end panels there was only one set of bottom laterals, which was in the plane of the shoe pins. All the top laterals, bottom laterals, and sway-bracing diagonals were of rods connected to the trusses by pins. A longitudinal strip of timber was placed on top of each line of stringers, and supported the wooden cross-ties, which, in turn, carried the rails.

At both ends of the bridge and viaduct the line had already been double-tracked, and, on account of the very heavy traffic and the increase in the weight of the loads carried, it became necessary to provide double track and, at the same time, increase the strength of the bridge. Various plans were considered, and the one finally adopted retained the same alignment as the old bridge, letting the new viaduct rest on the old footings and the new spans on the old piers.

Erection of Viaduct.

The new viaduct, designed in such a manner that it could be erected without interrupting traffic, was built in 1911. New footings had been placed under the old viaduct columns some years before. They were built on concrete piles, and each was designed to carry the load of a double-track viaduct column. The viaduct approach consisted of sixty-eight spans of 30 ft. 8¼ in., and one span of 61 ft. 4½ in., giving a total length of about 2 148 ft. New columns were provided throughout and new girders for all but twenty spans at the south end. For these twenty spans the old girders taken from the remainder of the viaduct were framed together in pairs, and two were used under each rail. These old girders were in good condition, and this substitution saved the expense of considerable new material. The viaduct runs through the sparsely settled part of the town, and its height varies from 20 ft. at the south end to 50 ft. at the north or river end.

FIG. 1.—THE OLD NORFOLK AND WESTERN RAILWAY BRIDGE AT KENOVA, W. VA.

FIG. 2.—THE NEW BRIDGE AT KENOVA, W. VA.

Erection was started at the north end and was carried on without interrupting traffic and without the use of falsework except under the first two spans. The traveler ran on rails laid over the new outside girders, and the first two spans had to be erected on falsework in order to have a place on which to erect the traveler. A track was laid on the ground alongside the viaduct for nearly its entire length, and the new material, as it was shipped in, was unloaded by a small derrick car near its proper place in the viaduct.

A derrick was placed on Pier 1, at the north end of the viaduct, and with it was erected the falsework and the first two new spans of the viaduct. The derrick then raised the traveler used in erecting the remainder of the viaduct. This traveler consisted of two wooden bents resting on wooden sills. The bents were shaped like those of an ordinary wooden gantry traveler, and were braced together. A platform, 21 ft. 6 in. above the rail, carried two hoisting engines which worked the booms on the forward end of the traveler. These booms were of steel, one 80 ft. and the other 65 ft. long. They turned on foot-blocks made fast to the front end of the sills, and each was capable of lifting about 12 tons. As there were no braces between the bents below 21 ft. 6 in. above the rail, trains were run through the traveler without interrupting its work in any way. The old viaduct being single-track, trains were carried on the two inside girders of the new work until the viaduct was completed. As a change of grade was found necessary in the new work, the Railway Company, some time before commencing the erection of the new steel, had jacked up all the old viaduct columns until the rail was at its proper level for the new work. Wood blocking was placed on top of the concrete pedestals to carry the old columns in their raised position. The new viaduct bents consisted of two columns connected by a cross-girder, on top of which rested four lines of longitudinal girders. The bents were connected by longitudinal bracing in alternate openings.

The method of erection (shown by Fig. 3) was as follows: The traveler erected two new bents on blocking level with the top of the concrete pedestals, one alongside each old bent. A temporary wooden strut was bolted to the old bents to brace them, and the old longitudinal bracing was removed. The ends of the old girders over these bents were fastened together and, using both booms of the traveler, the girders were hoisted a few inches, so as to clear the tops of the old

columns. This condition is shown by (1) of Fig. 3. The old bents were then pulled out to one side by lines from the traveler. New bents were pulled into place, and the old girders were slacked down on them, timber blocks on the tops of the columns being used to make up the difference in height of the old and new girders. Trains were then allowed to pass. The new longitudinal bracing was put in between the new bents, and the old bents were cut to pieces. This condition is shown by (2) of Fig. 3. The track was picked up at each end of the old girder span and blocked so as to clear the old

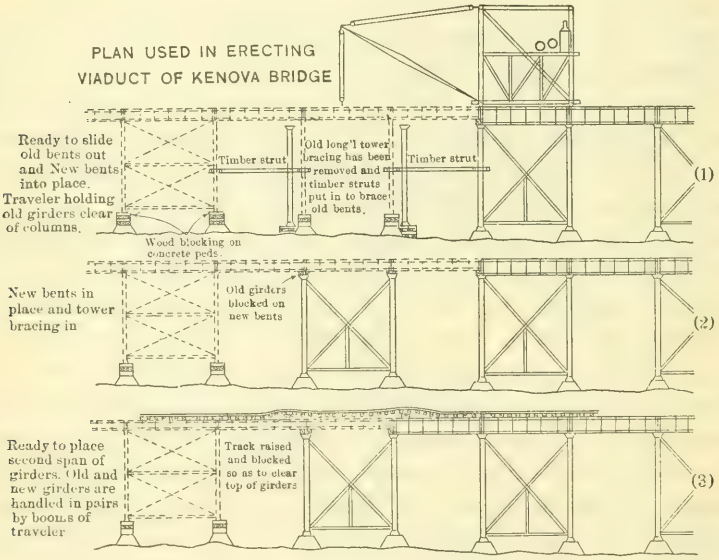

PLAN USED IN ERECTING
VIADUCT OF KENOVA BRIDGE
Ready to slide old bents out and New bents into place. Traveler holding old girders clear of columns.
Timber strut
Old long'l tower bracing has been removed and timber struts put in to brace old bents.
Timber strut
(1)
Wood blocking on concrete pedt.
New bents in place and tower bracing in
Old girders blocked on new bents
(2)
Ready to place second span of girders. Old and new girders are handled in pairs by booms of traveler
Track raised and blocked so as to clear top of girders
(3)

FIG. 3.

girders, the latter then being pulled out from under the track and laid on the ground below. The bracing between the girders was not cut out, so that, while one boom was removing the old girders, the other one hoisted the new girders into place. The new girders were shipped riveted up in pairs with bracing and cross-frames. The track was again picked up, the blocks taken out, the track slacked down, and trains allowed to pass. The second span of the old girders was handled in the same way. This is shown by (3) of Fig. 3. The traveler was then moved ahead two panels, and the operation was repeated.

None of these operations took more than 15 or 20 min., and it was very seldom that trains were held even a few minutes, as work

was not started if a train was seen to be approaching. The rate of progress was two spans, or about 60 ft. per day. The traffic, consisting mostly of coal trains, was very heavy during the reconstruction of the viaduct.

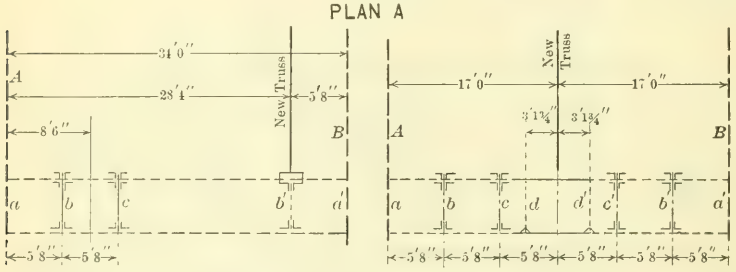
At the south end of the viaduct was the Kenova station, and this portion of the work was erected with a small derrick car, as the traveler would have interfered with the proper handling of passengers and trains. This portion of the old work was already double-tracked, and trains were allowed to run on one track while the derrick car erected the new work on the other. Traffic was then shifted, and the new steel for the second track was erected.

Various Schemes for Erection of New River Spans.

The railroad bridges over the Ohio River nearest to Kenova, are at Point Pleasant, about 60 miles above, and at Cincinnati, about 150 miles below that town, so that in the reconstruction of the river spans, it was important to take every precaution to prevent any accident that would necessitate going either to Point Pleasant or to Cincinnati to cross the river. Another condition which had to be considered was the stage of the Ohio River, the rise during floods amounting to as much as 70 ft. A third condition was the fact that provision had to be made for river navigation during the reconstruction. On account of these conditions and the very heavy traffic, the contract was awarded with the provision that traffic should be maintained at all times, and that no falsework should be used in the river.

Several designs for the river spans were worked up in the New York office of the American Bridge Company. The first design provided for the addition of a third center truss and the reinforcement of the two old outside trusses. New stringers were to be provided, but the old floor-beams were to be cut so as to use their ends and a new central portion which would connect to the new center truss. The floor-beam splice on each side of the new truss was to be designed so as to transmit shear but not moment, and by this means the live load transmitted to the outside trusses could be limited to a definite quantity by the location of the floor-beam splice, and thus not overstress the old trusses. Fig. 4 shows the details of this method.

The second design provided for placing falsework under the old stringers only and disconnecting the latter from the old floor-beams, so that the falsework would carry the old stringers, old track, and live load. New floor-beams were to be suspended from the old ones by rods, and new spans were to be built up around the old ones on brackets attached to the ends of the new floor-beams in suspended position. After the new spans were swung, the old ones were to be blocked up on them and then taken down. Finally, the new floor-beams were to be raised to their final position and new stringers added. Fig. 5 shows the details of this method.

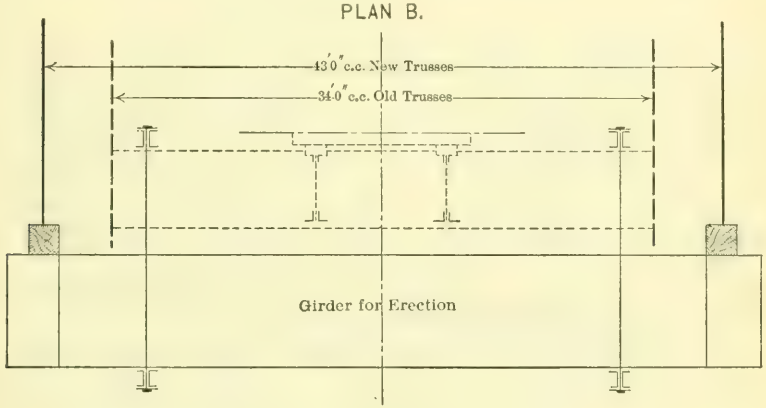


1.–Repair old trusses, rearrange old top laterals and portals to clear new truss when first erected, erect other new bracing.
2.–Erect new stringers at *b* and *c*.
3.–Remove old stringers and bolt one line to floor-beams at *b*′.
4.–Erect new truss at *b*′, swing it and remove old stringers.
5.–Move new truss to center line of bridge, connect it to transverse struts and erect new top laterals and portals.
6.–Block up stringers at any panel point on temporary erection girders hung from old trusses. Saw old floor-beam through at *d* and *d*′ and remove old center section. Place new center section, connecting it to truss and to old end sections of floor-beam, making efficient shear connections and top flange tension splices, but making butt joints for compression only in the bottom flange.
7.–Add reinforcing plates to floor-beams from *c* to *d* and *c*′ to *d*′.
8.–Erect new stringers at *b*′ and *c*′.

FIG. 4. Est. Wt. 9 000 000 lb.

The third design eliminated the use of falsework entirely. New stringers were to be placed on top of the old floor-beams and a track was to be laid so that the base of rail would be about 8 ft. 9 in. above the present base of rail and the center line of track about 10 ft. 0 in. from the original center line. Then the masonry piers under one span were to be cut down and the span was to be lowered and moved sidewise 5 ft. 9 in., the temporary track being shifted until it lined up with the track on the other spans. The other spans were then to be treated in the same manner. One new truss was to be erected in its permanent position on all spans and the truss swung; then the tempo-

rary track was to be shifted to its permanent location near the new truss. Brackets were to be attached to the ends of the old floor-beams, and one end of the latter was to be hung from the new truss just swung, so that the adjacent old truss would be clear of the masonry. The other new truss was then to be erected on the brackets in its permanent location and swung. The other ends of the old floor-beams were to be hung from the second new truss and the old spans taken down. Finally the new floor-beams and stringers were to be placed. Fig. 6 shows the details of this method.



1. Cut down old piers and place new pier girders.

2. Support old stringers on falsework, cutting them loose from floor-beams, and carry traffic on falsework during erection of new trusses.

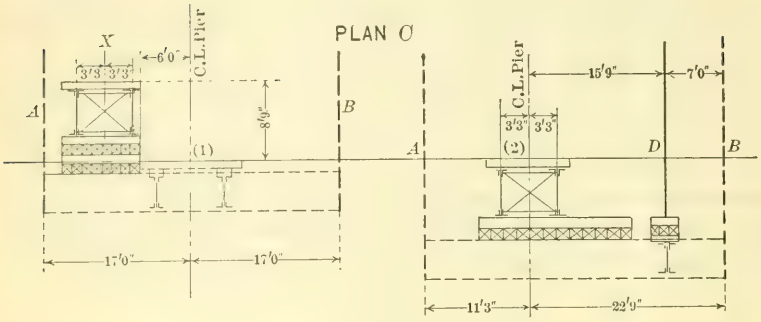
3. Erect new trusses outside of old trusses, as shown, connecting up bracing where possible.

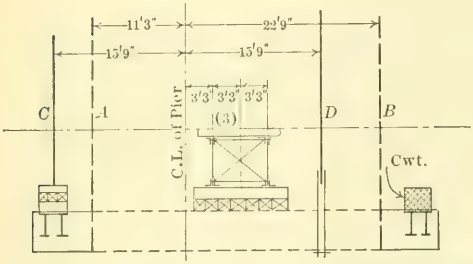
4. Hang erection girders from new trusses and take down old span, connecting up new floor and bracing throughout.

Est. Wt. 16 000 000 lb.

FIG. 5.

Each design had certain objectionable features. The first gave a patched-up bridge. The second gave a new bridge, but required falsework under all the spans. The third design provided for a new bridge and eliminated the falsework, but the erection programme was very complicated, requiring the lowering and sliding of the old spans and considerable shifting of the track.





1.–Cut down old masonry and place pier girders for one span.
2. Place a track at *X* on this span.
3.–Lower this span, move it to right and move track from *X* to (2) to line up with track at (1) on other spans.
4.–Proceed same with other spans.
5.–Erect truss *D* on all spans.
6.–Move track from (2) to (3) and shift traffic to it.
7.–Place brackets on truss *A* of one end span and hang up old floor-beams from *D*. Transfer weight of *B* to *D*, leaving *B* clear of masonry.
8.–Erect truss *C* of this span, connecting all bracing possible.
9.–Hang brackets from *C*, take down old span, connect up new floor and bracing.
10.–Proceed similarly with other spans.

Est. Wt. 13 000 000 lb.

FIG. 6.

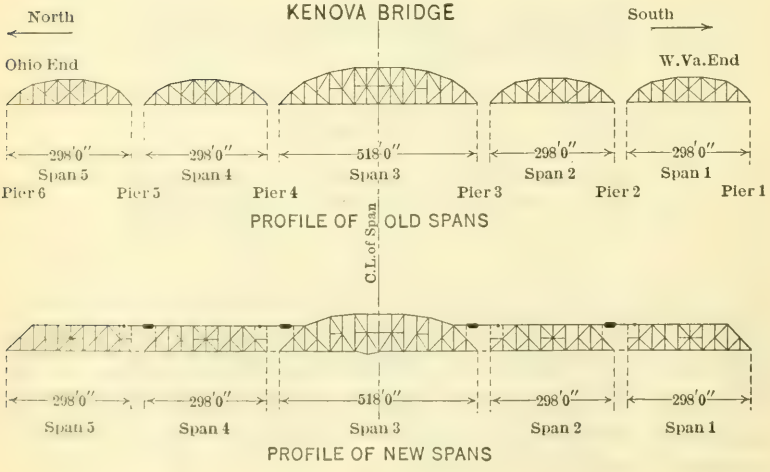


FIG. 7.

These objectionable features led to the adoption of the final design which, in brief, was as follows (Fig. 7): Spans 1 and 5 were to be erected according to the method of the second design. Then Spans 2 and 4 were to be erected around these old spans by cantilevering out from Spans 1 and 5, respectively. Span 3 was then to be erected around the old span by cantilevering each end out from Spans 2 and 4, and joining the two halves of Span 3 in the center. Finally, the old spans were to be blocked up on the new ones and removed.

Description of New Spans.

The span lengths on the new bridge were made the same as those on the old one, but the trusses were 43 ft. 0 in. from center to center, so as to straddle the old bridge completely. All the trusses were of the riveted type. The four center bottom-chord panels of Span 3, however, were made of eye-bars on account of erection conditions.

The live loads for the new bridge on each of the two tracks were as shown by Fig. 8, and were placed in such a position as to give maximum stress in the member considered.

The dynamic effect due to the passage of trains on the bridge was provided for by the impact formula, $I = \frac{300}{300 + L}$, in which

I = percentage of live load stress to be added for impact; and

L = length of loaded track producing the live load stress.

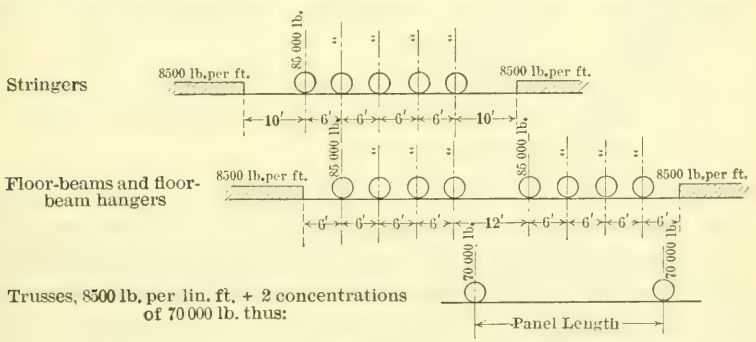


Fig. 8.

The structural steel for the new bridge had an ultimate tensile strength of from 62 000 to 70 000 lb. per sq. in., and the rivet steel an ultimate tensile strength of from 47 000 to 55 000 lb. per sq. in.

The allowable unit stresses were as follows:

Tension, 20 000 lb. per sq. in. (including dead, live, and impact stresses);

Tension, 24 000 lb. per sq. in. (including dead, live, impact, and wind and secondary stresses);

Tension in hangers, 16 000 lb. per sq. in.;

Compression, $\left(20\,000 - 100\,\frac{l}{r}\right)$ lb. per sq. in.;

Extreme fiber stress on pins, 30 000 lb. per sq. in.;

Shear on shop rivets and pins, 15 000 lb. per sq. in.;

Shear on air-driven field rivets, 14 000 lb. per sq. in.;

Shear on hand-driven field rivets, 12 000 lb. per sq. in.;

Shear on plate-girder webs, 12 000 lb. per sq. in.;

Bearing on shop rivets and pins, 30 000 lb. per sq. in.;

Bearing on air-driven field rivets, 28 000 lb. per sq. in.;

Bearing on hand-driven field rivets, 24 000 lb. per sq. in.;

Bearing on masonry, 600 lb. per sq. in.

After the bridge had been designed according to these loads and unit stresses, it was tested for Cooper's E-60 live load on each track, using the American Railway Engineering and Maintenance of Way Association Specifications for 1910, and the design was modified accordingly where the latter conditions required the larger sections. It was generally found that the first condition governed for the web members, and that Cooper's E-60 loading governed for the chords.

The assumed wind loads were:

30 lb. per sq. ft., on the loaded span;

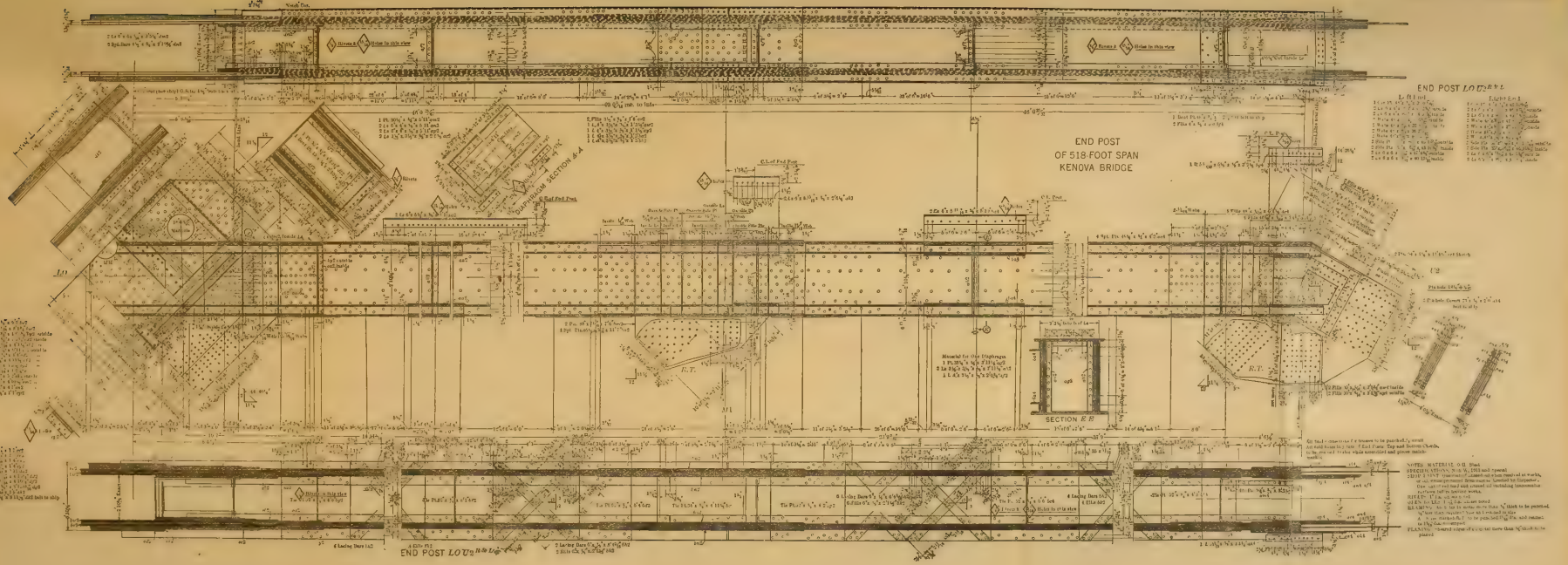
50 lb. per sq. ft., on the unloaded span after erection; and

50 lb. per sq. ft., during erection.

The unit stresses previously mentioned were used for testing the various members for erection stresses, and the only cases where it was found necessary to increase the sections on account of erection stresses were in the bottom chord members of the 298-ft. spans, which took heavy erection compression, namely, the four end panels at the river end of Spans 1 and 5 and the four end panels at both ends of Spans 2 and 4.

The rails on the new bridge are 100 lb. per yd. on the north-bound and 85 lb. per yd. on the south-bound tracks. The new ties are 8 by

PLATE I.
PAPERS, AM. SOC. C. E.
JANUARY, 1915.
GROVE AND TAYLOR
ON RECONSTRUCTION OF KENOVA BRIDGE.



10 in., notched to 9½ in. Each track was assumed to weigh 450 lb. per lin. ft. The assumed total dead loads per linear foot of bridge were 9 400 lb. for Spans 1, 2, 4, and 5, and 12 950 lb. for Span 3; and the actual total dead loads per linear foot of bridge were 9 380 lb. for Spans 1 and 5, 9 510 lb. for Spans 2 and 4, and 12 340 lb. for Span 3. All rivets are ⅞ in. in diameter, except those in the trusses of Span 3, where 1-in. rivets are used. Fig. 9 is a diagram of the 298-ft. spans and Table 1 is the stress sheet. Fig. 10 is a diagram of the 518-ft. span and Table 2 is the stress sheet.

Special attention should be called to the fact that the trusses of the 518-ft. span are the largest riveted trusses ever built for a simple truss span.

Plate I shows the details of the end post for the 518-ft. span. Fig. 11 shows a typical floor-beam of this span.

Remodeling the Tops of the Piers.

The length of the two piers under Span 3 was 46 ft. 0 in. under copings, and of the other four 43 ft. 0 in. As the new trusses were made 43 ft. 0 in. from center to center, so that they would clear the old spans, the shoe bearings of the new trusses came directly over the ends of the piers. It was necessary, therefore, to place distributing girders under the new trusses in order to transfer the reaction of these trusses in toward the center of each pier, and hence the piers had to be cut down to receive these pier girders. Two devices were used in order to reduce this pier cutting to a minimum. The rockers on the truss spans were made 36 in. deep, and narrow enough to be placed inside the bottom chord. Fig. 12 shows the position of the rockers of Span 3. Second, the pier girders consisted of three separate girders connected by diaphragms, and the cast-steel pedestals under the girders were made with ribs extending up between the girder sections and were connected to them by pins. In this way the pier cutting was reduced by about 6 ft., and it was only necessary to remove five courses of masonry, including the coping stones.

Fig. 13 shows the remodeled top of Pier 3. To form a neat finish to the tops of the piers after they were cut down, and also to give a strong bond, the Railway Company put a reinforced concrete coping around the three masonry courses immediately under the new steel. After it was determined just how many courses were to be removed,

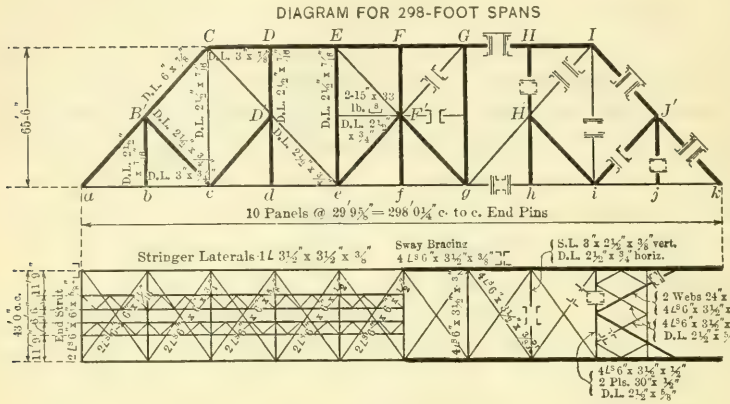


DIAGRAM FOR 298-FOOT SPANS
C D E F G H I
B B' D F F' H J'
a b c d e f g h i j k
65'-0"
10 Panels @ 29' 9⅝" = 298' 0¼" c. to c. End Pins
Stringer Laterals
Sway Bracing
S.L. 3" x 2½" x ⅜" vert.
D.L. 2½" x ¾" horiz.
2 Webs 24" x ½"
End Strut
43'0 c.c.
11'0"
2 Pls. 30" x ½"
D.L. 2½" x ⅝"

FIG. 9.

STRESSES, ETC., IN 298-FOOT SPANS.

END REACTION ON PIER.	END REACTION ON SPAN.		DEAD LOAD, IN POUNDS PER LINEAR FOOT.	
D = 797 000 lb.	D = 690 000 lb.	D = 690 000 lb.	Trusses	6 550
L_1 = 1 404 000 "	L_1 = 1 260 000 "	L_2 = 939 000 "	Floor	1 950
I_1 = 468 000 "	I_1 = 420 000 "	I_2 = 313 000 "	Track	900
2 669 000 lb.	2 370 000 lb.	1 942 000 lb.	Total	9 400
at 600 = 4 448 sq. in.				

C. S. Pedestal 3 ft. 8 in. by 8 ft. 0 in. 100 lin. ft. of 36-in. rockers.

PIER GIRDERS.

Shear.	*Moment.*
D = 715 000 lb.	D = 42 740 000 ft-lb.
L = 1 260 000 "	L = 75 600 000 " "
I = 420 000 "	I = 25 200 000 " "
2 395 000 lb.	143 540 000 ft-lb.
at 12 000 = 199.6 sq. in.	at 20 000 = 7 177 sq. in.

Web. 3 Pls. 46 × ⅝ = 86.3
6 Fills. 34 × 15/16 = 165.8
6 R. Pls. 44 × ⅝ = 165.0
417.1 sq. in.

Top Flange. 6 Ls. 6 × 6 × 15/16 = 54.5g. = 44.7n.
12 Covers 15 × 1 1/16 = 123.8g. = 90.8n.
178.3g. = 135.5n.

Bottom Flange. 6 Ls. 6 × 6 × 1 8/16 = 54.5 sq. in.
9 Cov. 13 × ¾ = 87.8 " "
142.3 sq. in.

FLOOR-BEAMS.

Shear.	*Moment.*
D = 43 400 lb.	D = 7 047 000 ft-lb.
L = 343 000 "	L = 61 700 000 " "
I = 246 000 "	I = 44 300 000 " "
632 400 lb.	113 047 000 ft-lb.
at 12 000 = 52 sq. in.	at 82.5 = 1 370 000 at 20 000 = 68 5 sq. in.

Web. 82 × 1 1/16 = 56.4 sq. in.
⅛ web = 7.0 " "
2 Ls. 6 × 6 × ¾ = 16.9g. = 13.9 sq. in. n.
5 Cov. Pls. 16 × 1 1/16 = 55.0g. = 46.0 " " n.
68.9 sq. in. n.

STRINGERS.

Shear.	*Moment.*
D = 6 700 lb.	D = 599 000 ft-lb.
L = 127 000 "	L = 9 800 000 " "
I = 115 400 "	I = 8 920 000 " "
249 100 lb.	19 319 000 ft-lb.
at 12 000 = 20.8 sq. in.	at 65.8 = 294 000
Web. 69 × ½ = 34.5 sq. in.	at 20 000 = 14.7 sq. in.
	⅛ web = 4.3 " "
	2 Ls. 6 × 6 × ½ = 11.5g. = 10.5 " " n.
	14.5 sq. in.

10 Int. stiffr. 4 × 3½ × ⅜ crimp.

TABLE 1.—STRESS SHEET FOR 298-FOOT SPANS.

Member.	D.	L_1.	I_1.	$D + L_1 + I_1$.	L_2.	I_2.	$D \times L_2 \times I_2$.	W_1.	Sections for Spans 2 and 4.
a B'	− 838	− 1 701	− 569	− 3 108	− 1 269	− 428	− 2 532	8 610 000 ± 8.5	1 Cov. Pl. 38 × 13/16, 2 Ls. 6 × 4 × 3/4 4 Webs. 42 × 13/16, 2 Ls. 6 × 6 × 3/4 227.2 2 Pls. 29 × 1/2
B' C	− 745	− 1 511	− 506	− 2 762	− 1 099	− 368	− 2 212	8 610 000 ± 4.3	1 Cov. Pl. 38 × 13/16, 2 Ls. 6 × 4 × 3/4 4 Webs. 42 × 13/16, 2 Ls. 6 × 6 × 3/4 198.2
C D E	− 752	− 1 522	− 510	− 2 784	− 1 082	− 362	− 2 196	± 13 C D ± 32 D E	1 Cov. Pl, 38 × 13/16 = 30.9 2 Ls. 6 × 4 × 3/4 = 13.9
E F G	− 784	− 1 585	− 531	− 2 900	− 1 195	− 400	− 2 379	± 43	2 Ls. 6 × 6 × 3/4 = 16.9 4 Webs 42 × 11/16 = 115.5 177.2
a b c	+ 564	+ 1 145	+ 383	+ 2 092	+ 854	+ 286	+ 1 704	± 41 a b ± 118 b c	4 Pls. 48 × 3/4 = 144.0 g. = 126.0 n.
c d e	+ 564	+ 1 164	+ 390	+ 2 118	+ 856	+ 287	+ 1 707	± 169 c d ± 206 d e	4 Ls. 6 × 6 × 3/4 = 33.8 g. = 27.8 n. 133.8 n.
e f g	+ 784	+ 1 585	+ 531	+ 2 900	+ 1 195	+ 400	+ 2 379	± 224	
C D'	+ 373	+ 874	+ 399	+ 1 646	+ 646	+ 295	+ 1 314	.....	4 Pls. 30 × 5/8 = 75.0 g. = 65.0 n.
D' e	+ 280	+ 789 − 219	+ 360 − 137	+ 1 429 − 169	+ 605 − 168	+ 276 − 105	+ 1 161 − 86	....	4 Ls. 4 × 4 × 5/8 = 18.4 g. = 15.9 n. 80.9 n.
E F'	+ 47	+ 281 − 185	+ 141 − 103	+ 469 − 257	+ 217 − 145	+ 109 − 81	+ 373 − 195	....	2 Pls, 22 × 5/8 = 27.5 g. = 22.5 n. 4 Ls. 4 × 4 × 5/8 = 18.4 g. = 15.9 n. 38.4 n.
F' g	− 47	+ 185 − 281	+ 103 − 141	+ 257 − 469	+ 145 − 217	+ 81 − 109	+ 195 − 373	....	
C c	+ 227	+ 634	+ 354	+ 1 215	+ 452	+ 324	+ 1 003	.....	4 Pls. 30 × 5/8 = 75.0 g. = 65.0 n. 4 Ls. 4 × 4 × 5/8 = 18.4 g. = 15.9 n. 80.9 n.
E e	− 83	+ 137 − 208	+ 76 − 104	+ 158 − 395	+ 107 − 161	+ 60 − 81	+ 112 − 325	....	2 Pls. 26 × 1/2 = 26.0 4 Ls. 4 × 4 × 1/2 = 15.0 41.0
Sub-Hanger	+ 89	+ 343	+ 246	+ 678	+ 258	+ 185	+ 532	.....	2 Pls. 26 × 5/8 = 32.5 g. = 27.5 n. 4 Ls. 6 × 4 × 9/16 = 21.2 g. = 19.0 n. 46.5 n.
Sub-Diagonal	− 93	− 231	− 166	− 490	− 174	− 125	− 392	.....	2 Pls. 20 × 9/16 = 22.5 4 Ls. 4 × 4 × 1/2 = 15.0 37.5
Sub-Post	− 48	0	0	− 48	0	0	− 48	.....	2 Pls. 26 × 3/8 = 19.5 4 Ls. 4 × 4 × 3/8 = 11.4 30.9

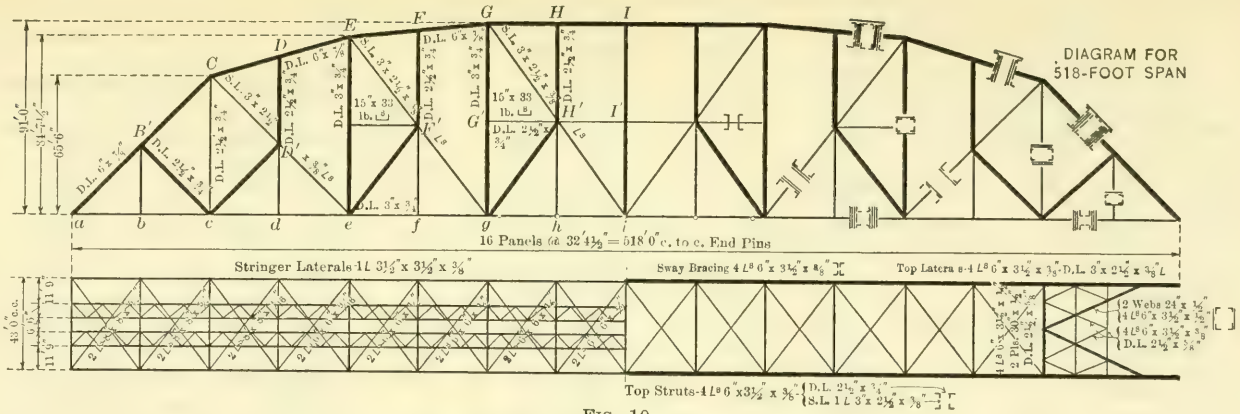


DIAGRAM FOR
518-FOOT SPAN
16 Panels @ 32′ 4½″ = 518′ 0″ c. to c. End Pins
Stringer Laterals 1 L 3½″ x 3½″ x ⅜″
Sway Bracing 4 L 6″ x 3½″ x ⅝″
Top Laterals 4 L 6″ x 3½″ x ⅜″ D.L. 3″ x 2½″ x ⅜″ L
Top Struts 4 L 6″ x 3½″ x ⅜″
D.L. 2½″ x ¾″
S.L. 1 L 3″ x 2½″ x ⅜″
12 Webs 24″ x ½″
4 L 6″ x 3½″ x ½″
4 L 6″ x 3½″ x ⅝″
D.L. 2½″ x ⅝″

FIG. 10.

STRESSES, ETC., IN 518-FOOT SPAN.

END REACTION ON PIER.

D = 1 852 000 lb.
L = 2 349 000 "
I = 529 000 "

4 730 000 lb.
at 600 = 7 883 sq. in.

END REACTION ON SPAN.

D = 1 712 000 lb.	D = 1 712 000 lb.
I_1 = 2 190 000 "	L_2 = 1 600 000 "
I_1 = 493 000 "	I_2 = 368 000 "
4 395 000 lb.	3 680 000 lb.

DEAD LOAD, IN POUNDS PER LINEAR FOOT.

Trusses	10 050
Floor	2 000
Track	900
Total	12 950

C. S. Pedestal 6 ft. 0 in. by 9 ft. 0 in.
183 lin. ft. of 36-in. rockers.

PIER GIRDER.

Shear.	*Moment.*
D = 1 740 000 lb.	D = 83 600 000 ft-lb.
L = 2 190 000 "	L = 105 000 000 " "
I = 493 000 "	I = 23 600 000 " "
4 423 000 lb.	212 200 000 ft-lb.
at 12 000 = 351.9 sq. in	at 20 000 = 10 610 sq. in.

FLOOR-BEAMS.

Shear.	*Moment.*
D = 47 000 lb.	D = 7 730 000 ft-lb.
L = 374 000 "	L = 67 300 000 " "
I = 261 000 "	I = 47 000 000 " "
682 000 lb.	122 030 000 ft-lb.
at 12 000 = 56.8 sq. in.	at 82.5 × 20 000 = 74.0 sq. in.

Web. = 82 × ¾ = 6.15 sq. in.

Flange. ⅛ web. = 7.7
2 Ls. 6 × 6 × ¾ = 16.9g. = 13.9n.
5 Cov. Pls. 16 × ¾ = 60.0g. = 52.5n. 74.1n.

STRINGERS.

Shear.	*Moment.*
D = 8 100 lb.	D = 788 000 ft-lb.
L = 134 000 "	L = 11 470 000 " "
I = 120 900 "	I = 10 350 000 " "
263 000 lb.	22 608 000 ft-lb.
at 12 000 = 21.8 sq. in.	at 65.8 × 20 000 = 17.2 sq. in.

W. = 69 × ½ = 34.5 sq. in. ⅛ web. = 4.3 sq. in.
2 Ls. 6 × 6 × ⅝ = 14.2g. = 12.9 sq. in. n.
Full T 38 ft. B, 34 ft., 29 ft., 25 ft., 19 ft.

Web. 3 Webs. 55 × ⅝ = 103.1
6 Fills. 39 × ⅞ = 204.8
6 Pls. 53 × ⅝ = 198.8 506.7 sq. in.

Top Flange. 6 Ls. 8 × 8 × ⅞ = 79.4g. = 61.7n.
12 Cov. Pls. 18 × 11/16 = 148.5g. = 111.4n.
173.1n.

Bottom Flange. 6 Ls. 8 × 8 × ⅞ = 79.4g.
9 Cov. Pls. 16½ × ¾ = 111.4g. 190.8g.

TABLE 2.—Stress Sheet for 518-Foot Span.

Member.	D_1.	L_1.	I_1.	$D_1 + L_1 + I_1$.	L_2.	I_2.	$D_1 + L_2 + I_2$.	W_1.	Sections.		
a B'	− 2 210	− 3 083	− 693	− 5 986	− 2 254	− 518	− 4 982	17 000 000	1 Cov. Pl. 48 × ¾	2 Webs. 48 × ½	
								± 477	4 Ls. 6 × 4 × 11/16	2 Webs. 48 × 9/16	
B' C	− 2 070	− 2 872	− 645	− 5 587	− 2 063	− 476	− 4 609	17 000 000	4 Ls. 6 × 6 × 11/16	4 Pls. 35 × 11/16	
								± 377	4 Webs. 48 × 11/16		
										Total 423 sq. in.	
C D E	− 2 000	− 2 793	− 627	− 5 420	− 1 963	− 462	− 4 425	± 170 C D	1 Cov. Pl. 48 × ¾	4 Webs. 48 × 11/16	
								± 104 D E	4 Ls 6 × 4 × 11/16	4 Pls. 35 × 11/16	
									4 Ls. 6 × 6 × 11/16		
										Total 321 sq. in.	
E F G	− 2 240	− 3 118	− 702	− 6 060	− 2 156	− 509	− 4 905	∓ 35 E F	1 Cov. Pl. 48 × ¾	4 Webs. 48 × 11/16	
								∓ 4 F G	4 Ls. 6 × 4 × 11/16	2 Webs. 48 × ⅜	
									4 Ls. 6 × 6 × 11/16	4 Pls. 35 × 11/16	357 sq. in.
G H I	− 2 380	− 3 222	− 746	− 6 448	− 2 264	− 535	− 5 179	∓ 33 G H	1 Cov. Pl. 42 × ¾	4 Webs. 48 × 11/16	
								∓ 46 H I	4 Ls. 6 × 4 × 11/16	2 Webs. 48 × ½	
									4 Ls. 6 × 6 × 11/16	4 Pls. 35 × 11/16	369 sq. in.
a b c	+ 1 550	+ 2 165	+ 487	+ 4 202	+ 1 582	+ 364	+ 3 496	± 286 a b	4 Ls. 6 × 6 × 13/16	= 36.4 g. = 29.9 n.	
								± 449 b c			
c d e	+ 1 550	+ 2 193	+ 492	+ 4 235	+ 1 589	+ 370	+ 3 509	± 597 c d	4 Webs. 48 × ¾	= 144.0 g. = 126.0 n.	
								± 713 d e	2 Webs. 48 × 13/16	= 78.0 g. = 68.2 n.	224.1 n.
e f g	+ 2 000	+ 2 808	+ 632	+ 5 440	+ 1 957	+ 463	+ 4 420	± 810 e f	4 Ls. 6 × 6 × 13/16	= 36.4 g. = 29.9 n.	
								± 882 f g	4 Webs. 48 × ¾	= 144.0 g. = 126.0 n.	
									2 Webs. 48 × 13/16	= 78.0 g. = 68.2 n.	
									2 Pl. 35 × 13/16	= 56.9 g. = 50.4 n.	274.5 n.
g h i	+ 2 304	+ 3 236	+ 726	+ 6 266	+ 2 220	+ 524	+ 5 048	± 907 g h	10 bars. 16 × 2	= 320.0	
								± 932 h i			
C D'	+ 673	+ 1 128	+ 293	+ 2 094	+ 882	+ 219	+ 1 714	.....	4 Ls. 6 × 6 × 5/8	= 28.4 g. = 23.4 n.	
									2 Webs. 30 × ½	= 30.0 g. = 26.0 n.	
									4 Webs. 30 × 9/16	= 67.5 g. = 58.5 n.	
											107.9 n.

TABLE 2—(*Continued*).

Member.	D_1.	L_1.	I_1.	$D_1 + L_1 + I_1$.	L_2.	I_2.	$D_1 + L_2 + I_2$.	W_1.	Sections.		
D' e	+ 525	+ 1 070	+ 290	+ 1 885	+ 800	+ 217	+ 1 542	.....	4 Ls. 6 × 6 × 5/8 = 28.4 g. =	23.4 n.	
		− 372	− 211	− 233	− 280	− 158	− 88	.....	4 Webs. 30 × 1/2 = 60.0 g. =	52.0 n.	
									2 Webs. 30 × 7/16 = 28.3 g. =	22.8 n.	
										98.2 n.	
E F'	+ 512	+ 1 043	+ 305	+ 1 860	+ 764	+ 230	+ 1 506	.....	4 Ls. 6 × 6 × 5/8 = 28.4 g. =	23.4 n.	
		− 334	− 164	− 157	− 256	− 129	− 43	.....	2 Webs. 30 × 1/2 = 30.0 g. =	26.0 n.	
F' G	+ 380	+ 985	+ 301	+ 1 666	+ 725	+ 223	+ 1 328	.....	4 Webs. 30 × 7/16 = 52.5 g. =	45.5 n.	
		− 467	− 214	− 427	− 362	− 167	− 276				
										94.9 n.	
G H'	+ 257	+ 853	+ 287	+ 1 397	+ 623	+ 216	+ 1 096	.....	4 Ls. 6 × 6 × 5/8 = 28.4 g. =	23.4 n.	
		− 494	− 199	− 522	− 375	− 153	− 359	.....	6 Webs. 30 × 7/16 = 78.8 g. =	68.3 n.	
										91.7 n.	
H' i	+ 128	+ 804	+ 283	+ 1 215	+ 609	+ 216	+ 953				
		− 621	− 238	− 773	− 475	− 183	− 572				
C c	+ 336	+ 687	+ 369	+ 1 394	+ 494	+ 265	+ 1 095	.....	4 Ls. 4 × 4 × 5/8 = 18.4 g. =	15.9 n.	
									4 Webs. 30 × 3/4 = 90.0 g. =	78.0 n.	
										93.9 n.	
E e	− 142	− 553	− 167	− 862	− 421	− 128	− 691				
		+ 494	+ 239	+ 638	+ 370	+ 183	+ 458	.....	4 Ls. 6 × 6 × 3/4 = 33.8		
G g	− 70	− 553	− 183	− 786	− 452	− 159	− 681	.....	2 Webs. 30 × 13/16 = 48.8		
		+ 563	+ 222	+ 739	+ 419	+ 171	+ 543				
									82.6		
I i	− 82	.....	.....	− 82	.....	.....	.....	.....	4 Ls. 4 × 4 × 3/8 = 11.4		
									2 Webs. 30 × 7/16 = 26.3		
									37.7		
Sub-Hanger	+ 127	+ 374	+ 261	+ 762	+ 264	+ 164	+ 555	.....	4 Ls. 6 × 4 × 9/16 = 21.2 g. =	19.0 n.	
									2 Webs. 30 × 5/8 = 37.5 g. =	32.5 n.	51.5 n.
B' C *C D'*	− 148	− 263	− 184	− 595	− 185	− 115	− 448	.....	4 Ls. 4 × 4 × 5/8 = 18.4 2 Webs. 22 × 5/8 = 27.5	45.9	
F' e	− 132	− 236	− 165	− 533	− 166	− 103	− 401	.....	do.		
H' g	− 128	− 230	− 160	− 518	− 162	− 101	− 391	.....	do.		
D D' *F F'-H H'*	− 82	.....	.....	.....	.....	.....	.....	.....	4 Ls. 4 × 4 × 3/8 = 11.4 2 Webs 30 × 7/16 = 26.3	37.7	

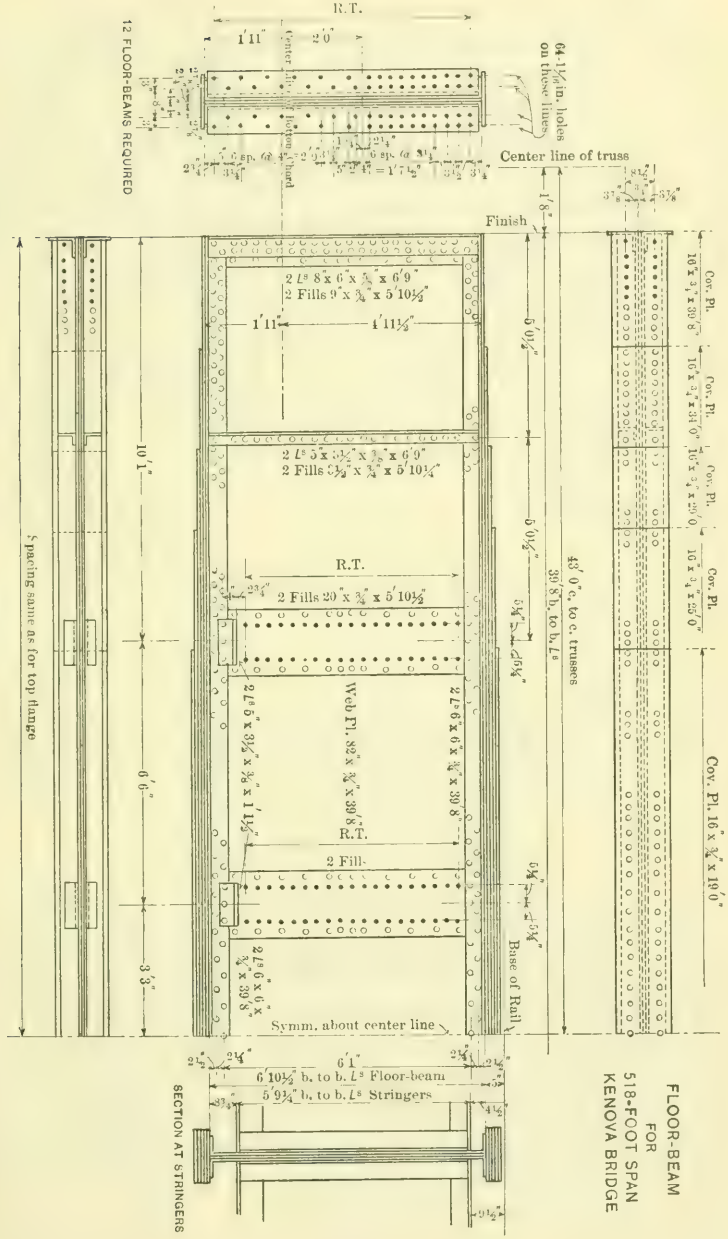

R.T.
1'11"
2'0"
12 FLOOR-BEAMS REQUIRED
64-1⅛ in. holes on these lines
Center line of truss
Finish
2 Ls 8"x 6"x ⅞"x 6'9"
2 Fills 9"x ¾"x 5'10⅛"
1'11"
4'11½"
5'0½"
2 Ls 5"x 3½"x ⅞"x 6'0"
2 Fills 3½"x ¾"x 5'10¼"
R.T.
2 Fills 20"x ¾"x 5'10⅛"
5¼"
5¼"
43'0" c. to c. trusses
39'8" b. to b. Ls
Web Pl. 39"x ⅜"x 39'8"
2 Ls 6"x 3½"x ⅜"x 1'1½"
2 Ls 6"x 6"x ⅜"x 39'8"
R.T.
2 Fill-
5¼"
5¼"
Base of Rail
Cov. Pl. 10"x ¾"x 39'8"
Cov. Pl. 10"x ¾"x 31'0"
Cov. Pl. 10"x ¾"x 25'0"
Cov. Pl. 10"x ¾"x 25'0"
Cov. Pl. 16"x ¾"x 19'0"
10'1"
6'6"
3'3"
Spacing same as for top flange
Symm. about center line
2¼"
6'1"
2¼"
6'10½" b. to b. Ls Floor-beam
5'9¼" b. to b. Ls Stringers
SECTION AT STRINGERS
FLOOR-BEAM FOR 518-FOOT SPAN KENOVA BRIDGE

Fig. 11.

this concrete coping was put on the piers up to the mark shown by the dotted line in Fig. 13, before the upper masonry courses were removed. The remainder of the coping, together with the concrete inside the pier girders, was placed after the new bridge was erected. The stone courses varied in thickness from 16 to 22 in., and the total depth removed averaged about 8 ft. The distance from the top of the pier girders to the bottom of the cast-steel pedestals was about 6 ft. 0 in. for the two large girders and about 5 ft. 3 in. for the eight small ones. The distance from the base of rail to the top of the pier girders was about 7 ft. 8½ in., and was constant for all girders. When the distance from the bottom of the cast-steel pedestal to the top of the masonry was small, loose plates were used to fill this space, and I-beam grillages, as shown by Fig. 13, were used when there was sufficient depth.

The concrete coping extended to a minimum of 16 in. from the pier face, consequently, the new caps are 15 ft. 6 in. by 49 ft. 9 in. for the two center piers and 13 ft. 6 in. by 46 ft. 6 in. for the other four. Three pairs of 2-in. bolts, 15 ft. 0 in. long, were placed in each pier, one pair in the center and one at each end. A plate, 6 in. by 1 in. by 4 ft. 3 in., connected the two bolts of each pair near each end, and was held in place by two nuts. A third nut clamped a 6 by ½ by 6-in. plate against the pier face. Fig. 13 shows the reinforcing bars. The lower set of vertical ¾-in. square bars extended above the first portion of the concrete coping put on the piers, and formed the bond when the remainder of the concrete was added.

Pier Girders and Material Track.

Each pier girder is made up of three separate girders with 1-in. continuous plates on top, under each truss and each end stringer. The cast-steel pedestals under the small girders have two verticals ribs, and those under the large ones have four vertical ribs, two inside and two outside, as shown by Fig. 13. The pins connecting the pedestals to the girders are 18 in. in diameter. The cast-steel pedestals are 3 ft. 8 in. by 8 ft. 0 in. for the small girders, and 6 ft. 0 in. by 9 ft. 0 in. for the large ones, and the distances from center to center of bearings are 33 ft. 0 in. and 35 ft. 0 in., respectively. Figs. 9 and 10 show sections of the girders. The girders, pins, and pedestals were assembled in the shop, and the total shipped weight was 88 tons for the small

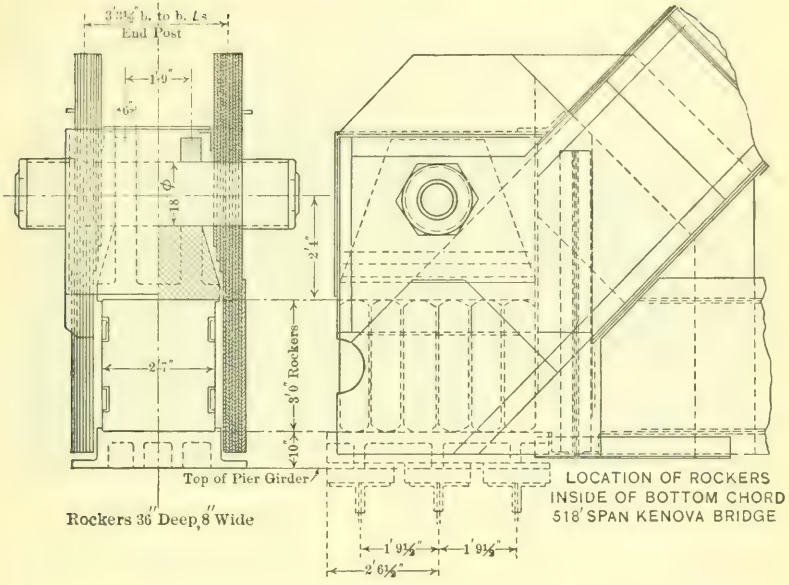

End Post
1'9"
6"
18"
2'4"
8'0" Rockers
2'7"
10"
Top of Pier Girder
Rockers 36" Deep, 8" Wide
LOCATION OF ROCKERS
INSIDE OF BOTTOM CHORD
518' SPAN KENOVA BRIDGE
1'9½"
1'9½"
2'6½"

FIG. 12.

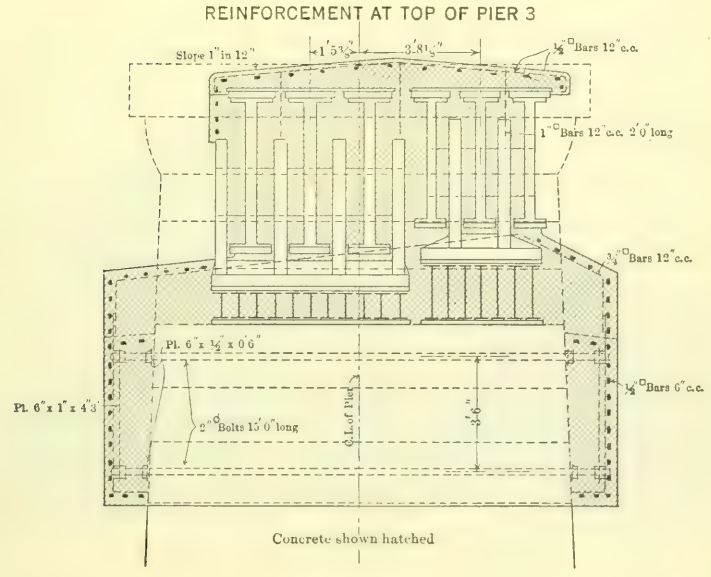

REINFORCEMENT AT TOP OF PIER 3
Slope 1" in 12"
1'5¾"
3'8½"
½" □ Bars 12" c.c.
1" □ Bars 12" c.c. 2'0" long
¾" □ Bars 12" c.c.
Pl. 6" x ½" x 0'6"
½" □ Bars 6" c.c.
Pl. 6" x 1" x 4'3"
2" □ Bolts 15'0" long
℄ of Pier
3'0"
Concrete shown hatched

FIG. 13.

girders and 127 tons for the large ones. The large girders were shipped on the special gun-carriage cars of the Pennsylvania Railroad. Fig. 14 shows one of these large pier girders on the cars. The deflection of the cars is quite noticeable.

The new spans were erected around the outside of the old ones, and the new floor-beams were erected in a temporary position, suspended from the new bottom chords, so that they would clear the old floor-beams. As it would have been very difficult to hoist the new material from the main track through the old bracing, and also on account of the heavy traffic, temporary material tracks were provided outside the new trusses. Temporary brackets were attached to the ends of the new floor-beams and on these the new stringers were laid, thus providing outside tracks which were entirely independent of the operation of the trains on the center line of the bridge. At each end of the bridge these were connected to the main track by timber trestles.

Method of Reconstruction of River Spans.

The reconstruction of the river spans was naturally divided into five separate operations:

(1) Erection of the pier girders;
(2) Erection of Shore Spans 1 and 5;
(3) Cantilever erection of Spans 2 and 4;
(4) Cantilever erection of Span 3; and
(5) Change of new floor to its final position.

1.—Erection of the Pier Girders.—The pier girders had to be placed before the work on the new spans could be commenced. The piers had to be cut down about 8 ft., and the girders placed without interruption of traffic. Furthermore, no falsework could be used to support the old spans while this work was going on. This problem was solved by the use of a steel gallows-frame, the general method of erection being indicated by Fig. 16. The frame consisted of two heavy steel posts connected at the top by a box-girder, which was high enough to allow trains to pass under it. The posts rested either on the end pins of the old spans, or on shoes placed on blocking on each side of the old shoes. The frame was stiffened by knee-braces extending from the posts up to the portal box-girder. From pins at the top of the posts, 16 by 2-in. eye-bars hung, and to the lower end of these eye-bars were fastened links which held the end pins of the span to be supported.

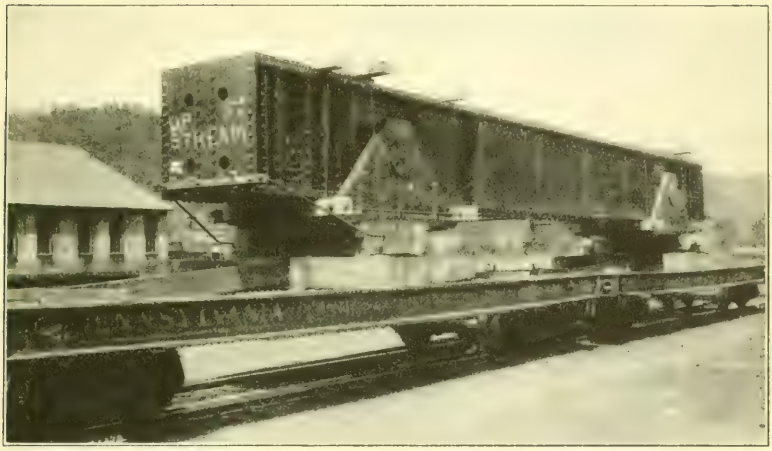


FIG. 14.—LARGE PIER GIRDER.

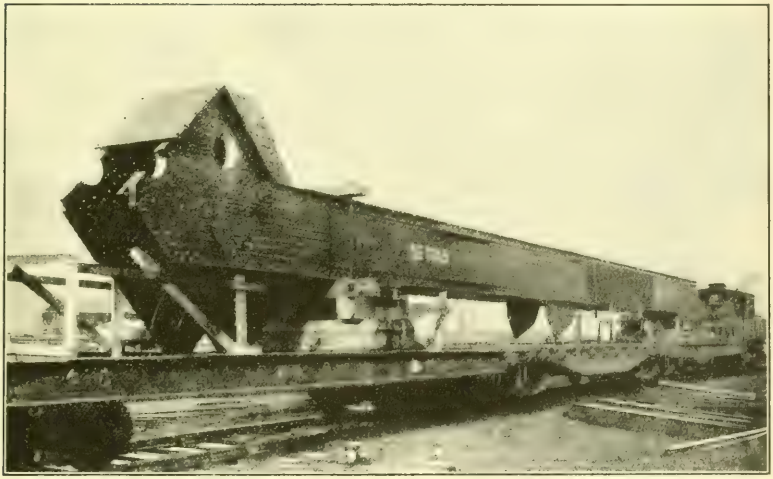
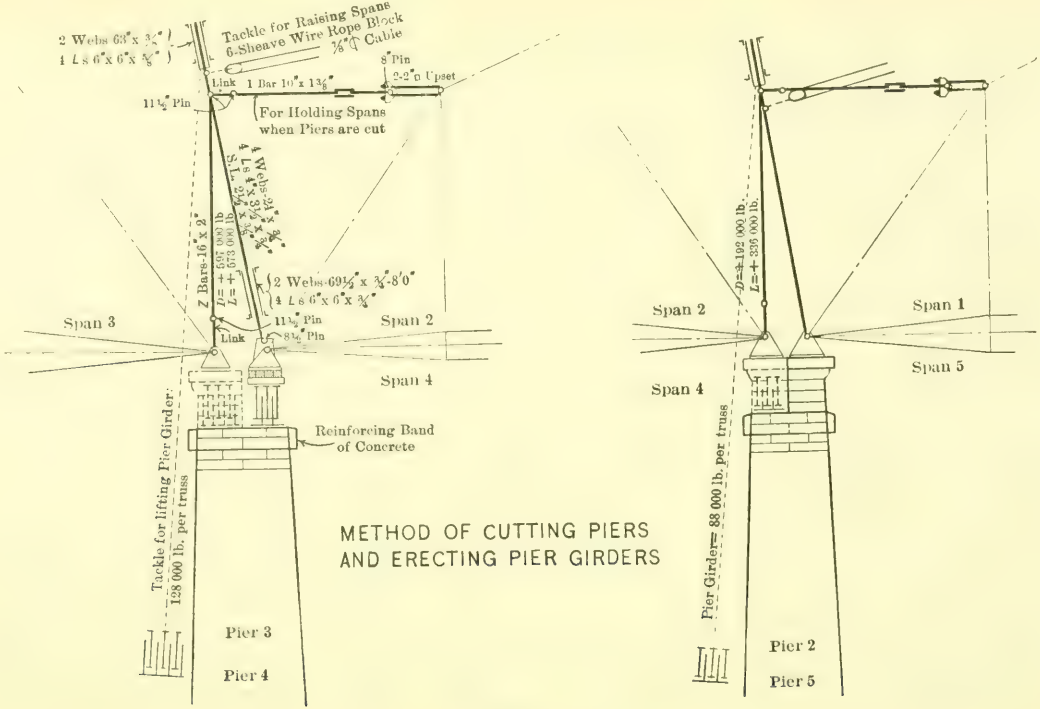


FIG. 15.—END POST FOR SPAN 3.



METHOD OF CUTTING PIERS AND ERECTING PIER GIRDERS

FIG. 16.

From the pin at the top of the post, a 10 by 1⅜-in. horizontal adjustable eye-bar extended back to an equalizer which, in turn, connected to the pin at the hip point of the anchor span.

A set of six-sheave, wire-rope falls was connected to the top of the gallows-frame at one end and to the second top-chord panel point at the other. The lead line from these falls ran to an electric hoisting engine placed near-by on the old bridge. By tightening up on the falls, the shoes of the old span were lifted clear of the masonry. As there were certain adjustments to be made in the elevation of the base of rail, it was the intention to use the falls to raise the span to the proper elevation and then disconnect them and use the adjustable eye-bar for anchorage. Any subsequent adjustment was to be made by the turn-buckle, the threads on the eye-bars being about 15 in. long. The adjustable bar was only used once, however, namely, over Pier 2; in the remaining cases the falls were used for anchorage, and the adjustable bar was discarded.

After the span was raised to the proper elevation, the masonry was removed, course by course, from one-half of the pier until the proper depth was reached, as shown for Piers 2 and 5, on Fig. 16. This cutting was done by the Railway Company, and the time for cutting down half a pier averaged about two weeks. When a train approached, blocking was inserted in place of the masonry cut away, and the span was slacked down on this blocking to allow the train to pass. The gallows-frame was designed to carry live load as well as dead load, and, in fact, did so at the beginning of the cutting of Pier 1, but the Railway Company afterward decided to use blocking during the passage of trains. This necessarily impeded progress, as much better time could have been made had the pier cutting been carried on independently of the operation of trains. When one-half the pier had been cut down, the top was leveled off to receive the girder. The cars containing the girder were run out on a car-float and brought alongside the pier, about 85 ft. below the final position of the girder. The portal girder of the gallows-frame was also designed to carry the weight of the pier girders. Two sets of six-sheave, ¾-in., wire-rope falls fastened to the portal girder of the gallows-frame were hooked to the pier girder below. Three-sheave, manila-rope falls were attached to the old span and used to keep the girder clear of the masonry as it was hoisted. The manila-rope falls were worked by a derrick car run out

on the bridge and the wire-rope falls by the electric engine used to raise and lower the gallows-frame. This engine had four drums and eight spools. The wire-rope falls were 19 ft. 0 in. apart, wide enough to straddle the track and yet sufficiently narrow to fall inside the trusses. Trains were stopped while the girder was hoisted, the blocking being kept under the old shoes until the girder reached the pier level. Then the top of the gallows-frame was pulled back a little, so that the blocking could be removed and the girder swung into place. The top of the girder was about 2 ft. below the bottom of the old shoe, so that blocking was placed on which the old span would rest after being slacked down. As there were no end floor-beams, the end stringers of one span were supported by pockets attached to those of the adjacent span while the piers were being cut. The gallows-frame was then reversed to the other span on the same pier, and the remainder of the pier cutting was done, as shown for Piers 3 and 4, on Fig. 16.

Where it was impracticable to let the gallows-frame posts rest directly on the end pins, a special cross-girder was designed to fasten to the post. This girder, which was wide enough to straddle the old end post, rested on two small shoes, and was used at Piers 1, 3, 4, and 6. At Piers 1 and 6 (the end piers), it was necessary to erect a double timber bent to support the gallows-frame, and as there was no adjacent span at these piers to use for anchorage, the falls at Pier 1 were attached to the viaduct columns and those at Pier 6 to anchor rods fastened to the abutments of a road crossing about 300 ft. from the latter pier. Fig. 17 is an end view of Span 1, showing the gallows-frame with the cross-girders at the bottom of the post and the falls fastened back to the viaduct. It also shows the falls used to lift the pier girder. The pier girders for Span 1 were delivered on shore and skidded to the bottoms of the piers. The remaining pier girders, however, were brought to the foot of their respective piers on floats, as mentioned previously. Fig. 19 shows the pier girder for Pier 5 being lifted into place.

Work was started on Pier 1, and Piers 2, 3, 4, 6, and 5 were completed in succession. No difficulties of any moment were experienced, except that it proved a more difficult job than was expected to take down the gallows-frame from one pier and erect it on another. This was due partly to the old spans being in the way and partly to having to work between trains.

2.—Erection of Shore Spans 1 and 5.—After the pier girders were placed under Span 1, the erection of the new work on this span was started. It was originally intended to erect Spans 1 and 5 simultaneously, then Spans 2 and 4, and finally Span 3, the two erection gangs working independently of each other. As the piers for Span 1 were ready for the new steel long before those for Span 5, it was impossible to carry out this part of the programme. Fig. 21 shows the general method of erection of Spans 1 and 5.

Falsework was erected beneath the span, as shown by Fig. 21, and the new floor-beams were brought out and hung from the old ones by 3-in. rods. As these floor-beams were erected, they were blocked temporarily on falsework so as not to put too much load on the old span while it was carrying traffic. Pony bents were erected on each side of the new floor-beams, and the old stringers were blocked up on these bents. The rivets connecting the old stringers to the old floor-beams were cut out, letting the falsework carry the live load, track, and stringers. The hanger rods holding up the new floor-beams were then screwed up until the latter showed the right amount of camber, which had been determined previously. The conditions governing this will be given later.

I-beams were placed on the top chord in the two center panels and blocked up at the panel points so as not to allow any bending in the top chord from the weight of the traveler. The track was laid on these I-beams and the small traveler was erected, a 6-ton ginny-wink being used. This traveler consisted of a 30 by 34-ft. frame, with one 16-in. steel wheel at each corner and one in the center of each sill; 12 by 12-in. posts were placed at each corner and braced together. A lifting beam, composed of two 24-in., 100-lb. I-beams connected by diaphragms, was put at each end of the traveler on top of the posts. One 50-ft. boom was placed at each end of the traveler, resting on a foot-block in the center of the cross-sill. A four-drum, eight-spool, 100-h.p., electric engine was placed on the floor of the traveler, which floor consisted of seven 15-in., 42-lb. I-beams resting on the traveler sills. After the traveler was erected, it put in the remainder of the falsework on top of the span.

The new stringers were erected on the brackets on the ends of the floor-beams, and temporary tracks were laid on these stringers throughout the entire length of the span, thus providing means for bringing the

FIG. 17.—GALLOWS FRAME AT PIER 1.

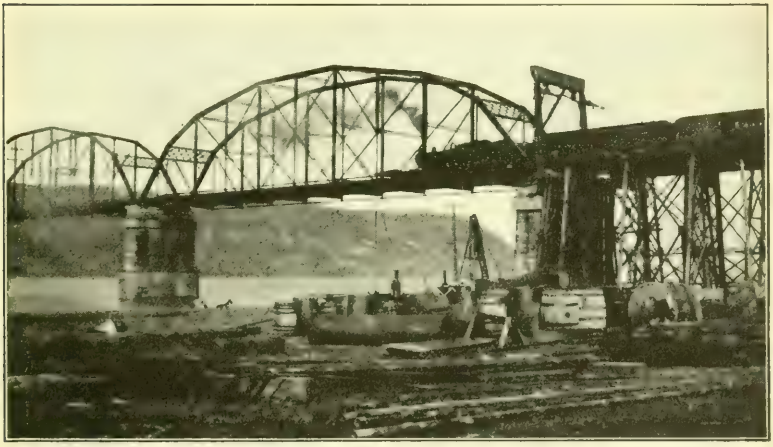


FIG. 18.—GALLOWS FRAME HOLDING SPAN 1.

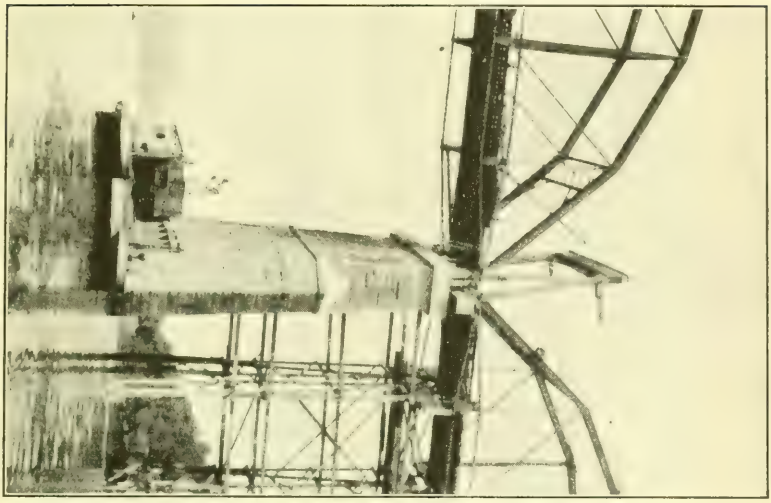


FIG. 19.—PIER GIRDER BEING LIFTED FROM BARGE.

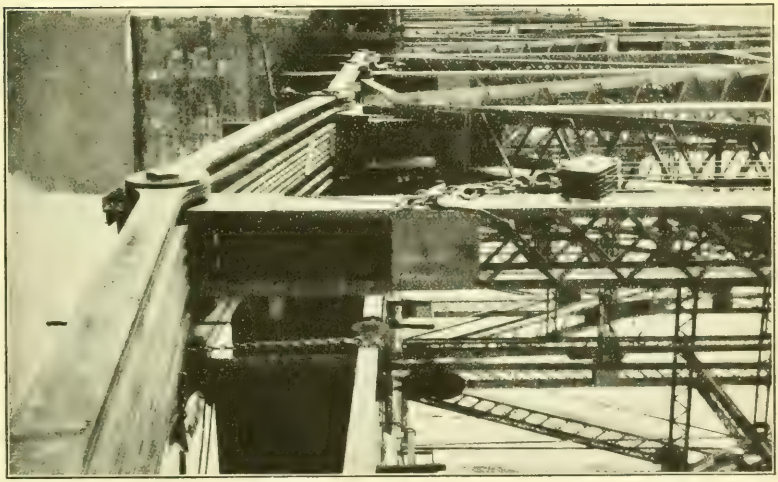


FIG. 20.—PANEL POINT *j* BEFORE CLOSING.

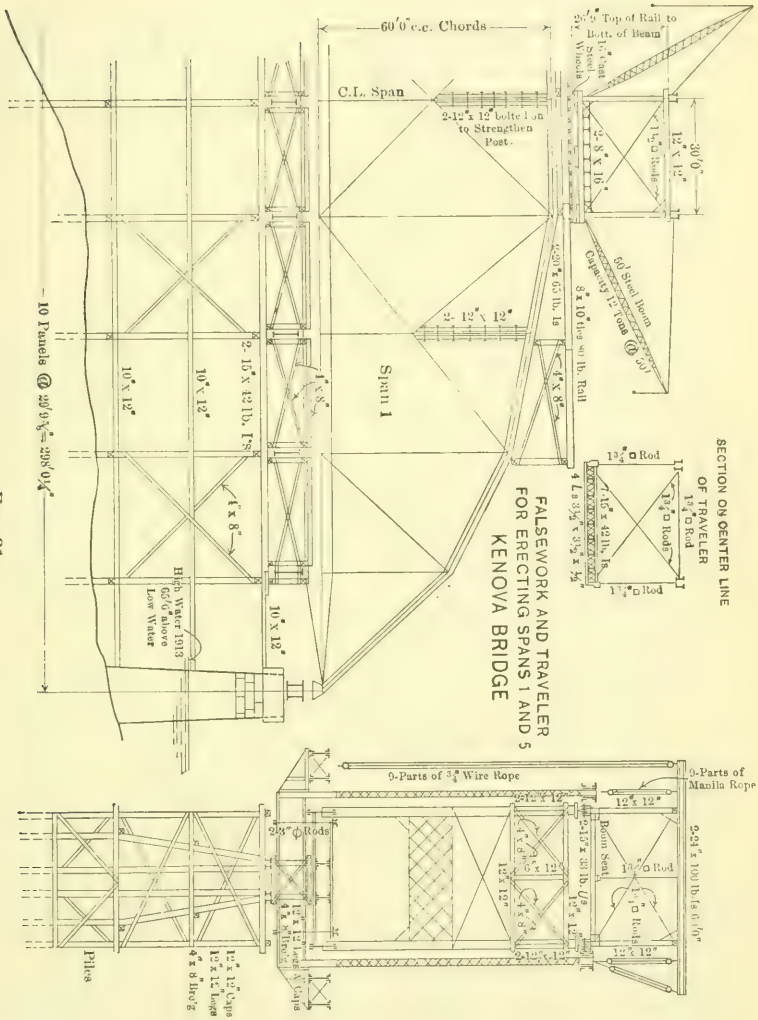

FALSEWORK AND TRAVELER
FOR ERECTING SPANS 1 AND 5
KENOVA BRIDGE
60'0" c.c. Chords
C.L. Span
Span 1
10 Panels @ 29'0"= 290'0"
SECTION ON CENTER LINE
OF TRAVELER
Piles
9-Parts of 3/4" Wire Rope
9-Parts of
Manila Rope
Boom Steel
High Water 1913
66'0" above
Low Water

FIG. 21.

new material out to its proper place in the bridge entirely independent of the main track. Run-offs from these material tracks connected with the viaduct about 100 ft. from Pier 1. All the bottom chords were run out on this track and put in place, and then all the web members. On each end of each lifting beam of the traveler were two sets of falls, one wire-rope set over the center of the material track and one manila-rope set over the center of the new truss. The material was lifted to the proper height by the wire-rope falls and then fleeted over to place by the manila-rope falls. The end pieces of the bottom chord were lifted with one set of falls on the corner of the traveler and outhauled into place. As the traveler was not allowed to stand in the middle of the panel, as it put too much load on the I-beams, practically every piece of bottom chord had to be outhauled. On account of the heavy concentrations coming over the sub-panel points, the old sub-posts had to be strengthened. This was done by shoring them with 12 by 12-in. timber, as indicated on Fig. 21. The new bottom laterals were placed in the plane of the top flange of the new floor-beams in their temporary position.

After all the new bottom chords and web members were erected, the new floor-beams were blocked up on the outside legs of the falsework so that the additional load of the top chords and end posts would be taken directly by the falsework. This was done in order not to put too much load on the old span.

Starting at the river end of the span, the end posts were placed, then the top chords, and finally the end posts at the shore end. In lifting the end post, the traveler was placed at the hip of the span, and the falls lifted the upper end. A derrick-car standing on the main track handled the lower end of the member. The car had to be fastened down very securely, as the loads were so far to one side, and the old track stringers had to be shored up on the pier. These end posts weighed about 54 tons, and the lifts proved to be among the most difficult on the work, but were made without accident.

As mentioned previously, special precaution had to be taken in setting the 3-in. bolts carrying the new floor-beams. Referring to Fig. 22, the theoretical center line of the bottom chord of the new span was 1 ft. 10⅝ in. below that of the old span, making the theoretical distance between the old and new floor-beams 2 ft. 11 in. At the bottom of Fig. 22, the upper line for the old span represents the posi-

tion of the center line of the bottom chord after the stringers had been disconnected. The other line represents the position of the bottom chord when the old span carried the new one. The upper line for the new span was the center line of the bottom chord under no load, and the other line represents the position of the bottom chord after the new span was swung. Now, the distance between the old and the new floor-beams had to be such that when the old bottom

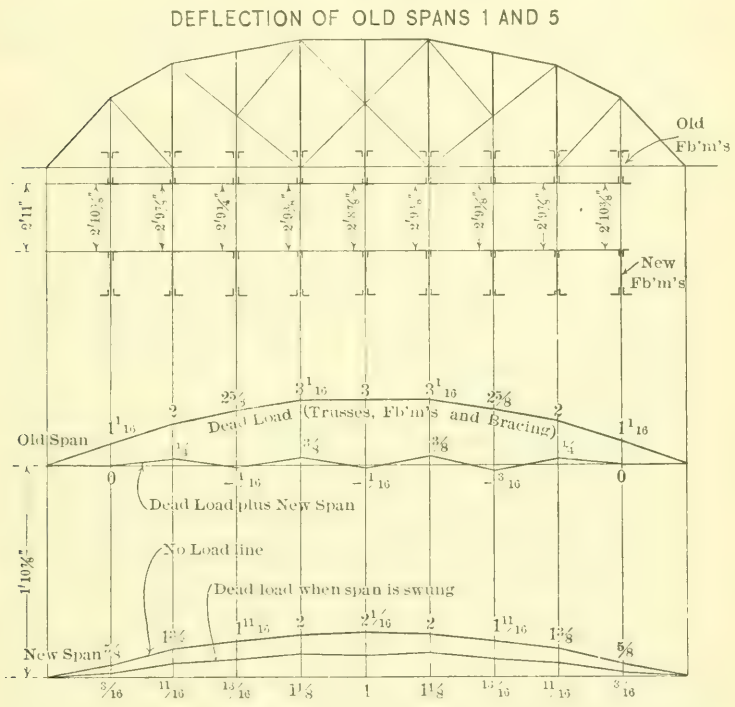


FIG. 22.

chord was in the position of the lower line for the old span, the new one would be in the position indicated by the upper line for the new span. This being the no-load line, the new span would be under zero stress and, consequently, all the truss connections could be easily made. The figures for placing the new floor-beams are given in Fig. 22. These figures were increased to allow for the settling of the wood blocking under the shoes of the old span.

The total actual deflection in the center of the span at the time the new floor-beams were blocked up on the outside legs of the falsework was about 2½ in. below the dead-load camber, or ½ in. below a horizontal line drawn from center to center of the end pins. Wedges had to be provided between the pony bents and old stringers so that the latter could be raised and lowered with the movements of the span, otherwise the old floor-beams would bear on the tops of the stringers and put more load on the falsework.

On account of the falsework on top of the old span being in the way, no new top laterals, portals, or sway bracing could be placed, so the new trusses were fastened temporarily to the old ones by rope lashing. The small traveler now removed this falsework, with the exception of the two center panels, at the same time putting in as much of the new portals as possible, one panel of the top laterals at each end, and the temporary sway bracing at the fourth and sixth panel points.

Jacking girders were attached to the bottom of each end post, and by using two 500-ton hydraulic jacks at each corner, the new span was jacked up until it was swung and the strain was taken off the 3-in. suspender rods. This distance was about 5 in. The old bottom chord had then gone back to the position of the upper line for the old spans in Fig. 22, and the new one assumed the position of the lower line for the new spans. The nuts on the 3-in. rods were unscrewed and the jacks lowered until the new span rested on the pier girders.

A traveler track was now laid on the top chord of the new span. This consisted of two 8 by 16-in. timbers on each truss, laid side by side, with a rail spliced to each. The small traveler then erected the sills and floor of the big traveler complete. After this was done, the small traveler was jacked up and extensions were put under it so that it could run on the track laid on the new top chord. The remainder of the falsework was removed from the top of the old span, and the remaining new top laterals and struts were placed. The old stringers were then reconnected to the old floor-beams by bolts, thus letting the old span again carry the live load. The small traveler placed a 12-ton, steel derrick with a 65-ft. boom on the deck of the large traveler. This derrick raised the remainder of the large traveler and was then taken down by the small traveler. The electric engine, which up to this time had been on the small traveler, was transferred

to the large traveler, and a 2-drum, 4-spool, steam engine was placed on the small traveler to do what little hoisting remained for this traveler. The large steel traveler was then rigged up complete. Fig. 23 shows sections of this traveler. The masts, booms, stiff legs, and cast-steel foot-blocks were standard parts of a 65-ton derrick, the remainder had been made especially for this bridge. The traveler could have been made somewhat lighter if it had been designed throughout for this bridge, but as the 65-ton derricks would be needed later for other work, it was decided to use them at Kenova as well. Aside from the great weight of the traveler (423 000 lb.) and its very great

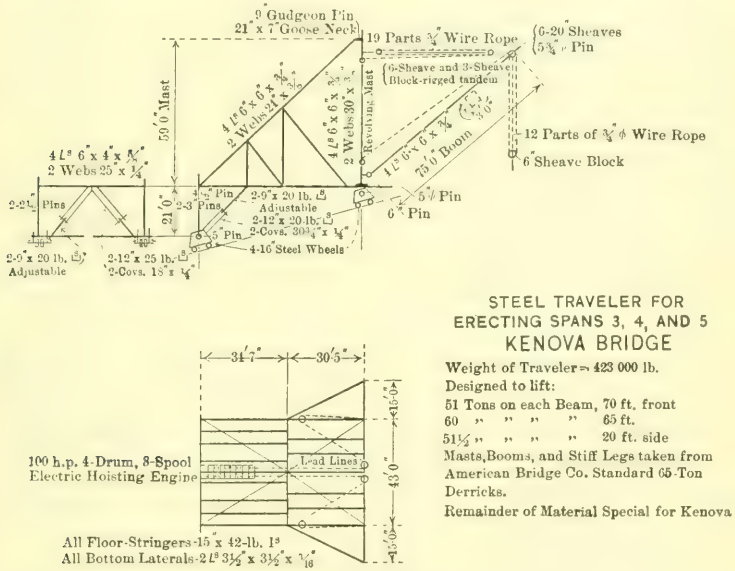

9" Gudgeon Pin
21" x 7" Goose Neck
19 Parts ¾" Wire Rope
6-20" Sheaves
5¾" Pin
6-Sheave and 3-Sheave Block-rigged tandem
50' 0" Mast
Revolving Mast
12 Parts of ¾" ϕ Wire Rope
6" Sheave Block
75' 0" Boom
5½" Pin
6" Pin
21' 0"
2-3 Pins
2-9" x 20 lb. Adjustable
2-12" x 20 lb.
2-Covs. 30¾" x ¼"
5" Pin
4-16" Steel Wheels
2-2½" Pins
2-9" x 20 lb. Adjustable
2-12" x 25 lb.
2-Covs. 15" x ¼"
STEEL TRAVELER FOR ERECTING SPANS 3, 4, AND 5
KENOVA BRIDGE
Weight of Traveler = 423 000 lb.
Designed to lift:
51 Tons on each Beam, 70 ft. front
60 " " " " 65 ft.
51½ " " " " 20 ft. side
Masts, Booms, and Stiff Legs taken from American Bridge Co. Standard 65-Ton Derricks.
Remainder of Material Special for Kenova
34' 7"
30' 5"
100 h.p. 4-Drum, 3-Spool Electric Hoisting Engine
Lead Lines
All Floor-Stringers-15" x 42-lb. I
All Bottom Laterals-2 L 3½" x 3½" x 7/16"

FIG. 23.

lifting capacity, the only other unusual feature was the arrangement for keeping the revolving mast plumb when working on the inclined top chords of Span 3. This was taken care of by a long leg in the back and telescoping braces. In order to raise the rear end of the traveler, the booms were hooked to the old span ahead. At the proper height, pins were driven in the back legs and telescoping braces. To work satisfactorily, a traveler or derrick with revolving masts should have these masts plumb.

The work of removing the falsework under the span was commenced, and the 15-in. I-beams, on which the pony bents had rested,

were placed on top of the new floor-beams. The old span was blocked up on these I-beams, and the old trusses and bracing were removed, using both travelers. The old floor system was not removed at this time, but was allowed to remain blocked up on the I-beams until all the new spans were erected. Span 5 was erected in the same way, but it was not started until some time after Span 1 had been put in place.

If there had been a flood in the river after the erection of the new trusses had been commenced, and this flood should have taken out the falsework, traffic would have been interrupted only for the length of time necessary to complete the erection of the new span, for then the weight of the new span could have been taken off the old one and trains allowed to run. The only loss would have been that of the falsework and the expense to the railway on account of the short time that traffic would have been interrupted. Had Spans 1 and 5 been erected by the straight-falsework method, using a gantry traveler, a wash-out would have meant a long interruption of traffic and the possible loss of most of the new span. It might be said, however, in favor of the latter method, that it was very unlikely that the river could have such a flood as to wash out this falsework, as most of Spans 1 and 5 are over dry land at the ordinary stage of the water. The latter method would have been cheaper, but by the one used, the Railway Company was sure of only a minimum interruption of traffic.

3.—Cantilever Erection of Spans 2 and 4.—The new Spans 2 and 4 were erected by cantilevering from the new Spans 1 and 5, respectively. It was originally intended to erect them simultaneously, but, actually, Span 2 was erected before Span 4.

The temporary erection ties, struts, and posts over Pier 2, which would connect the new Spans 1 and 2, shown by dotted lines in Fig. 7, were erected by the large traveler. The distance between the two hip points of adjacent spans was about 125 ft., and the ties were supported at each panel point by vertical posts. The total section in each tie was 225 sq. in., and, as the large traveler had to cross them, the ties were made deep enough to be good for this load in bending. The direct tension in each tie was about 4 000 000 lb. Span 1 acted as the anchor arm, and, for additional counterweight, 286 000 lb. of rails were placed on its shore end and the small traveler was run as far back toward the shore on this span as possible to serve also as a counterweight.

After the erection ties were in position, the end posts for Span 2, each weighing about 54 tons, were brought out on the material track and hoisted into place, one boom being used for each post. The shoes of these posts were put in in the material yard. The hitch for lifting them was placed so that they hung in the correct position after being hoisted clear of the push-cars. Temporary adjustable eye-bars, which fastened to the center of the end post and to the top of the erection post over Pier 2, were now connected up and held the end post until the remainder of the erection ties, the first piece of the bottom chord, and the first two panels of the web members, were put in by the traveler. After this was done, the adjustable eye-bars were removed. At the point where the erection ties were spliced, there were slotted pin-holes, and into each was inserted a round pin and a **D**-shaped plug.

A track was laid on the erection ties, and the traveler was moved out over Pier 2. The floor-beams with brackets attached were run out on the material track. With the brackets the over-all length was about 67 ft., and at first it seemed to be quite a problem to get these floor-beams under the new span. It was very easily done by the following method: The floor-beam was lifted off the car with one boom and slacked down to a barge on the river below the bridge. The barge was then pushed up stream by a tugboat, and both booms were hooked to the floor-beam, one at each end. It was then easily hoisted and fastened to the bottom chord. This method was used in placing all the floor-beams except those for Spans 1 and 5.

After the first two panels were erected, and the material track was laid, the remainder of the span was erected panel by panel, the large traveler creeping out on the top chord. As much as possible of the new bracing was placed at this time. The erection tie was shortened $7\frac{1}{4}$ in. in order to be sure that the river end of Span 2 would clear the pier girder; this clearance was about 3 in. A typical cross-section of Span 2, during and after erection, is shown at the right of Fig. 24.

Jacks were then placed under jacking girders attached to the shore end of Span 1 and the river end of Span 2, and both travelers were run over Pier 2 so as to put as little load as possible on the jacks. The jacks were pumped and the ends of the spans raised until the stresses in the trusses were reversed and there was zero stress in the

erection ties. At this time each span was acting as a simple span, and the **D**-shaped plugs previously mentioned were knocked out of the slotted holes. Each slot was so long that, after the spans were lowered, the erection ties could still be used for the travelers to cross from one span to the other. The total jacking was about 25 in., Span 1 being jacked 16 in. and Span 2, 9 in. This latter span could not be jacked any more on account of fouling old Span 2. During this jacking, the track was broken and the traffic interrupted, but the whole operation, from the time the track was broken until it was reconnected, did not take more than 2 hours. The spans were then lowered on the pier girders.

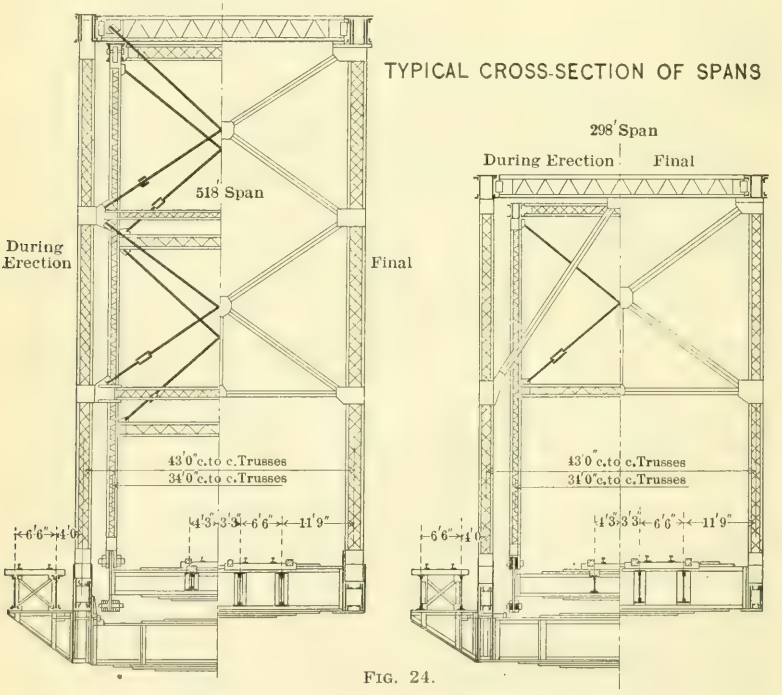
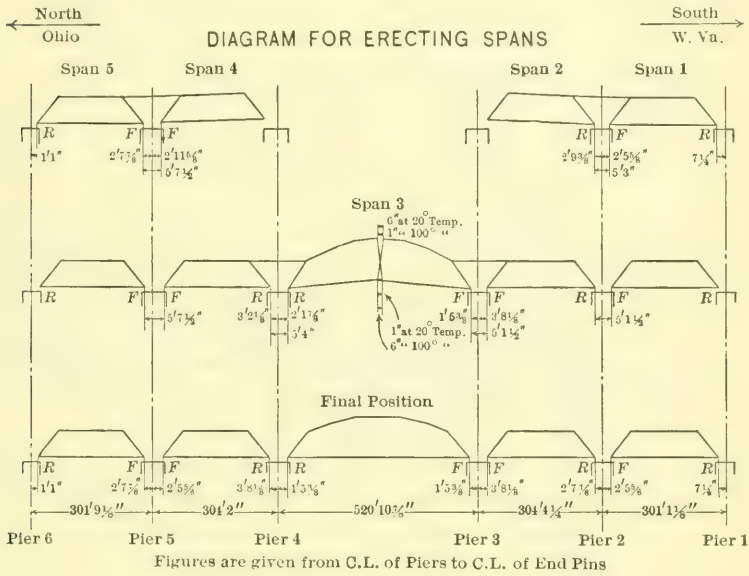


FIG. 24.

I-beams were placed on top of the new floor-beams of Span 2, and on these old Span 2 was blocked up, and then the trusses and bracing were taken down. These trusses were allowed to remain on new Span 2 so as to be used as a counterweight when the span acted as the anchor span during the erection of new Span 3. The method of erecting Span 4 was similar to that used for Span 2.

4.—Cantilever Erection of Span 3.—The small traveler crossed the erection ties to Span 2, removing the ties as it crossed. These ties were designed so that many of the pieces could be used again between Spans 2 and 3, and such as could be used were erected in position over Pier 3 by the large traveler. (See dotted lines in Fig. 7.) The small traveler was placed as near as possible to the shore end of Span 2 to serve as a counterweight, and an additional counterweight was provided by the rails which had previously been used on Span 1.



Figures are given from C.L. of Piers to C.L. of End Pins

FIG. 25.

Span 3 was erected by cantilevering each half from new Spans 2 and 4, and then joining the two halves in the center. This is shown on Fig. 25. The south end of Span 3 was fixed and the north end on rockers. The adjacent or south end of Span 4 was also on rockers, so that there were two rocker ends over Pier 4. The purpose of this will appear later. The joint in Span 3 was made at the center of the top chord, but at the sub-panel point next to the center in the bottom chord (see Fig. 7) the four center panels of the bottom chord were made of eye-bars. As the bottom chords shortened under erection compression, and as there would be still more contraction in cold weather, new Span 4 had been erected 6 in. south of its final position.

That was done so as to be sure that at a temperature of 20° there would be a minimum lap of 1 in. Furthermore, to insure a minimum gap of 1 in. in the top chord at 100°, the erection ties were shortened 9¼ in. over Piers 3 and 4. In order to close Span 3, the shore ends of Spans 2 and 4 were to be jacked until the top chords of Span 3 touched, closing the gap. Then, as jacking continued, the bottom chord would straighten out by the longitudinal movement of Span 4 and the north half of Span 3. On further jacking, the stresses would reverse, and finally all three spans would be acting as

SETTING OF ROCKERS AT PIER 4 FOR ERECTING SPAN 3

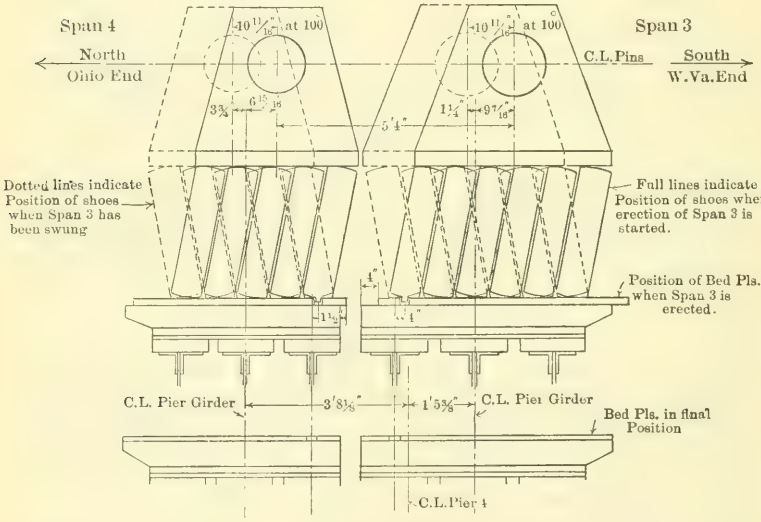


FIG. 26.

simple spans and Span 4 would have moved back to its final position. The rockers on Pier 4 were adjusted by sliding plates so that the inclination south before jacking would about equal the inclination north after Span 3 was closed. Fig. 26 shows the setting of the rockers over Pier 4.

After the erection ties were placed, the end post for Span 3 was brought out. This end post was about 105 ft. long and weighed 95 tons by itself, or about 105 tons when the shoe and pin were attached. It is shown in Fig. 15. As the capacity of each boom was only 65 tons, these posts had to be handled with both booms of the traveler. This was done by a balance beam hooked to the post at such a point

that, when the end post was lifted clear of the cars, it hung at the proper inclination. The balance beam served to distribute the load equally between the two booms. In order to place the end posts, the traveler had to stand directly over Pier 3, and, as the material track extended only as far as the pier, the booms could not reach the balance beam unless some provision was made for extending the material track farther out. Also, it was practically impossible to run the end post out on the main track and hoist it between the portals of old Span 3 and new Span 2. To get the end post in such a position that both booms could be hooked to it, two new floor-beams with brackets attached were brought out and suspended from the old floor-beams of Span 3 at the first and second panel points, using the 3-in. rods which were previously used similarly in Span 1. New stringers were placed on the brackets, and thus the material track was extended about 65 ft. As the connection of the brackets to the floor-beams was not sufficient to stand the load of the end post and the cars when the bracket was not connected to the new bottom chord, the brackets at the ends of the floor-beam were connected by 3-in. rods.

The end posts were shipped on three cars, the two on which the load rested being special gun-carriage cars of the Pennsylvania Railroad. These cars have no floor in the center, so that the gusset-plates at the ends of the end posts were allowed to project down through the open space in the center of the car. The posts could not be loaded on ordinary flat cars as the gusset-plates would have projected above the clearance lines. When the end posts arrived at Kenova, they were not unloaded, but one end was lifted high enough for the idler car to be taken out. The end car was then put back, but in such a position that the center of gravity of the end posts was as near as possible to the forward end of the car. The posts were brought out on the material track and placed by the traveler without any trouble. Temporary adjustable eye-bars, similar to those used during the erection of the end post for Span 2, were used.

After the first two panels of Span 3 were erected the other panels up to the center were erected by methods similar to those used for Span 2. The left side of Fig. 24 shows a typical cross-section of Span 3, during and after erection. In this case, however, the traveler had to climb the curved top chord. To pull the traveler up the inclined top chord of the new span, two sets of six-sheave, wire-rope falls were

used, fastened to the top of old Span 3. Also, two sets of four-sheave, manila-rope falls were put on, to catch the traveler in case anything gave away. After the traveler was pulled up to its proper place, the brackets were fastened to the top chord behind the front shoes. The falls were then slacked off, and the brackets prevented the traveler from running down the incline.

Attention has been called to the fact that the erection ties were shortened $7\frac{1}{4}$ and $9\frac{1}{4}$ in. over Piers 2 and 3, respectively, for certain purposes. There was another feature to be considered, which necessitated this shortening being kept to a minimum. The actual eleva-

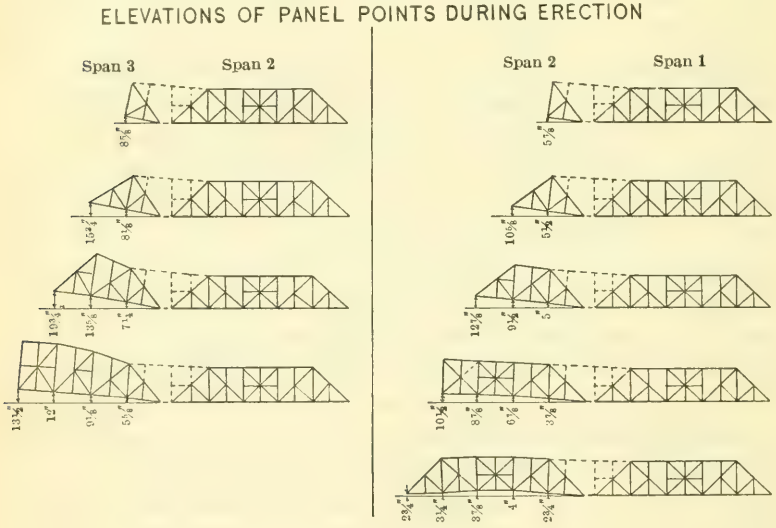


FIG. 27.

tions of the intermediate bottom-chord panel points of the cantilever spans were greater at the time the floor-beams were placed than after the span was erected prior to being swung. Thus, in Fig. 27, the sixth panel point of Span 2 was $12\frac{7}{8}$ in. above the horizontal when the floor-beam was placed, and only $3\frac{7}{8}$ in. just before the span was swung. Also, the sixth panel point of Span 3 was $19\frac{3}{4}$ in. above the horizontal when the floor-beam was placed, and only 12 in. just before the span was closed. The old Spans 2, 3, and 4, were carrying traffic during this period, therefore it was necessary to drop the new floor-beams sufficiently to clear the old spans. At the same time, it was

not advisable to go down too far, as the bottom lateral system in temporary position was in the plane of the top of the new floor-beams, and hence below the new bottom chords.

To determine as nearly as possible the actual deflections of the old spans under traffic, a set of levels was taken on the bottom-chord panel points of all five old spans, first under dead load and

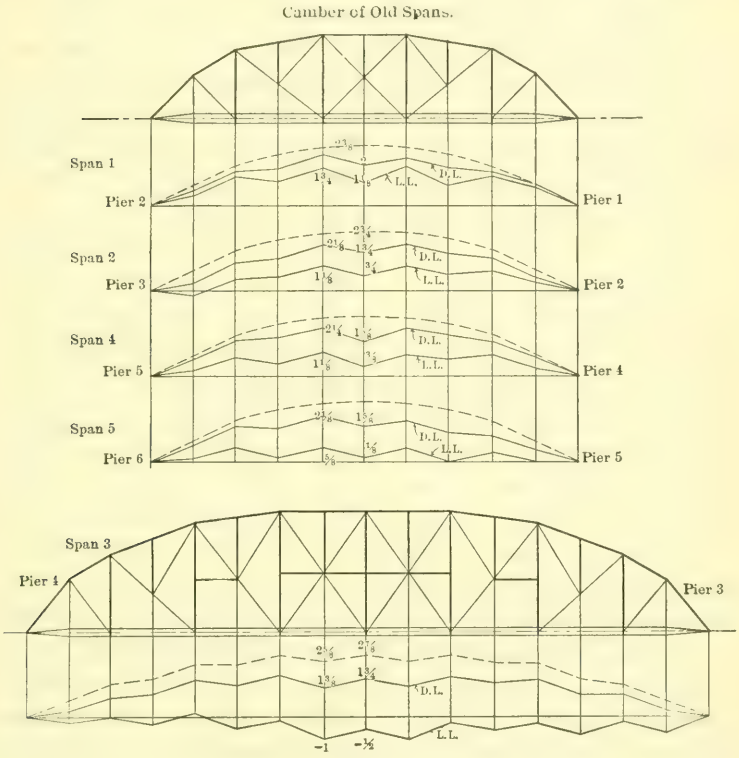


FIG. 28.

second under an actual live load, a train being stopped on the bridge for the purpose. The results of these tests are plotted in Fig. 28. The dotted lines represent the theoretical dead-load line, the upper full lines are the actual dead-load lines, and the lower full lines are the positions of the bottom chords under dead and live loads. It will be seen from Fig. 28 that although the bottom chord of the 298-ft. spans did not go below the horizontal under live load, the

518-ft. span near the center went down as much as 1 in. As a matter of precaution, therefore, the floor-beams near the center of the 518-ft. span were dropped from 4 to 8 in. more than had been intended originally. It was not necessary to place the three center floor-beams of Span 3 until after the new span was swung, as there was no material track in the four center panels of this span.

When all the trusses and bracing which could be placed before the span was closed, had been erected, the condition at the center was as shown in Fig. 29. There was actually a gap of about 10 in. at *I* when the last piece of the top chord was placed, and the lower end of the diagonal, *Ki*, lapped the gusset-plate at *i* about 6 in. more than it would in its final position. The point, *j*, was about 45 in. below its final position. The upper end of the hanger at *j* was held in a vertical position by a bolt which moved in guides so as to allow for a vertical movement of 4 ft. The bottom-chord members from *g* to *k* were eye-bars, with pins at *g*, *h*, *i*, *j*, and *k*. The panel load at *j* was about 60 000 lb., making the horizontal component of the stress in the members, *ij* and *jk*, about 240 000 lb. Referring to the central diagram of Fig. 25, both spans were fixed over Pier 3 and the rocker over Pier 4. The vertical reaction of Span 3 was about 2 000 000 lb. and, assuming a coefficient of friction of 20%, there were 400 000 lb. of friction to resist the 240 000 lb. pull from the suspended panel point, *j*. As both spans were on rockers at Pier 4, the horizontal pull at this pier had to be transferred to the fixed end of Span 4 over Pier 5. Here, the reaction was comparatively small, so that, in order to prevent Span 4 from moving, under the action of this horizontal force, it was necessary to fasten the anchor end of Span 4. Fig. 20 shows the drop in the bottom-chord eye-bars at *j* immediately before closing Span 3.

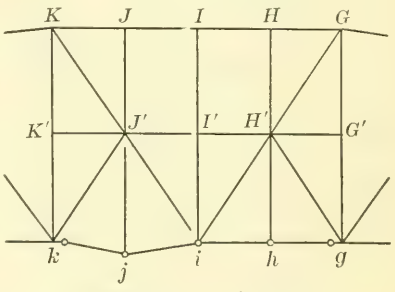


FIG. 29.

When everything was ready to close Span 3, traffic was interrupted at about 7 o'clock one morning and the track was broken. Jacks

were placed under jacking girders attached to the trusses at the shore ends of Spans 2 and 4 over Piers 2 and 5. As Span 4 had to move longitudinally about 6 in., the jacks under this span were placed on a carriage on rollers. These jacks were then pumped out. The top chords came together at *I*, then, as jacking was continued, the bottom chord straightened out, the hanger at *j* moved up until connection could be made at the top of the hanger, and the member, *Ki*, moved until it could be connected to the gusset-plate at *i*. Then, as the jacking proceeded, the stresses began to reverse, until finally all three spans were acting as simple spans, and there was zero stress in the erection ties. The D-plugs were then knocked out of the erection ties, the last plug being removed about 2 P. M., thus taking about 7 hours to swing Span 3. During all the jacking after the top chords had butted, Span 4 was moving north, the actual extent of this movement being about 8 in.

The total jacking was about 46 in. After Span 3 was swung, Spans 2 and 4 were lowered until they rested on the pier girders. Traffic was resumed about 6 P. M., after a delay of about 11 hours. This was the longest delay during the reconstruction of the bridge, and was the only serious one for the Railway Company. The entire closing was done without any serious difficulty. To insure uniform movement in jacking and lowering, it was necessary for the two jacks under each jacking girder to be connected in tandem.

I-beams were put on top of the new floor-beams of Span 3, and the old span was blocked up on these beams, and the old trusses and bracing were removed. The old trusses of Spans 2 and 4, which had been used for a counterweight, were also removed at this time.

5.—Change of New Floor to its Final Position.—This was the final stage of the erection. The large travelers were used to change the floor system, except a few panels at the shore ends of Spans 1 and 5, which could not be reached by the booms.

It had been planned to start simultaneously at each end of Span 3 and work toward the center, and, as traffic had to be stopped while a panel of floor was being changed, it was desired to change a panel with each traveler whenever an interval of time could be given. The wooden deck of the north traveler caught fire from the sparks of a train beneath, and as it was discovered late at night, the fire could not be put out quickly, and damaged the electric hoisting engine

so badly that two steam engines had to be substituted for it. This delayed the north traveler about 2 weeks, and made it impossible to start both travelers on the floor at the same time. The south traveler had changed the floor of the south half of Span 3 before the north traveler was repaired. From this time forward, however, when an interval was given to change the floor, each traveler would change a panel. On account of the heavy traffic, it was seldom that there was a chance to change more than one panel a day, and on some days no interval at all could be given. After the floors of Spans 2, 3, and 4 were changed, the erection ties over Piers 3 and 4 were taken down and re-erected over Piers 2 and 5. The travelers were then run back on Spans 1 and 5, and the floors changed with the travelers, except the five panels at the shore ends, which were changed by falls hooked to the new trusses.

The method used in changing a panel of floor was as follows: If the ends of the old stringers, in any panel of the span in which the change was to be made, did not clear the cover-plate of the new floor-beam by about 2 in., the old floor system was jacked on end until this condition was accomplished.

Two of the new stringers used in the material track were transferred to the inside of the trusses and hung from the truss by chains. The rivets connecting the old stringers to the old floor-beams were cut out and replaced by bolts, and those connecting the old stringer laterals to the stringers were cut out and replaced by one bolt in each connection. As many of the bolts fastening the new floor-beams to the hanger plates and the hanger plates to the bottom chord were removed as thought safe. Permanent lateral plates were fastened to the bottom of the new floor-beam, and the temporary lateral plates and angles on top of it were removed. The laterals at this point were supported by rope lashing attached to the bottom chord. A piece of rope lashing was run under all the ties in the panel to be changed. All this was done before the interval was given for the change.

As soon as the last train passed, rails were disconnected at each side of the panel, and this section was hauled on end to clear the panel. A line from the traveler was made fast to each end of the rope lashing under the old ties, and when the strain was taken on this line by the hoisting engine, all the ties in the panel were dumped

into the river below. These ties were in such bad shape that it did not pay to save them. Stringer laterals were removed, and the stringers were picked up with the booms of the traveler and outhauled back on the portion of the floor already changed, and piled at each side of the track.

A set of rope falls made fast to the sway bracing was now hooked to the old stringers in the next panel ahead, in order to hold them up after the old floor-beam was removed. As there was no convenient place to put the old floor-beam, it was hoisted up and hung by chains

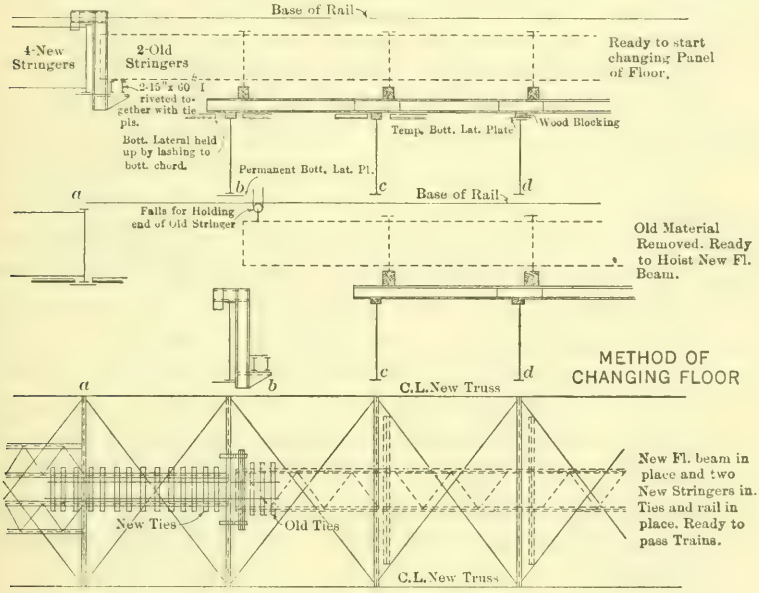

Base of Rail
4-New Stringers
2-Old Stringers
Ready to start changing Panel of Floor.
2-15" x 60" I riveted together with tie pls.
Bott. Lateral held up by lashing to bott. chord.
Temp. Bott. Lat. Plate
Wood Blocking
Permanent Bott. Lat. Pl.
a
b
c
d
Base of Rail
Falls for Holding end of Old Stringer
Old Material Removed. Ready to Hoist New Fl. Beam.
METHOD OF CHANGING FLOOR
C.L. New Truss
New Fl. beam in place and two New Stringers in. Ties and rail in place. Ready to pass Trains.
New Ties
Old Ties
C.L. New Truss

FIG. 30.

from the top of the span. On account of working inside the new trusses, it was hard to dispose of the old material taken out, as the new top bracing was in the way of the falls. I-beams in Panel *bc* (Fig. 30) were pulled back on end into Panel *cd*. The hangers were removed from Floor-beam *a* and put on Floor-beam *b* while in its suspended position. Falls from the traveler booms were then fastened to the ends of the new floor-beam to take the strain, and the remainder of the bolts connecting the floor-beam to the hanger plates and the hanger plates to the bottom chord were removed. The hanger plates were pulled out from between the floor-beam and the bottom chord

and the permanent filler plates were put in. These fillers extended above the top of the floor-beam in its final position, and were held in place by two bolts at the top of each plate.

Jacks, which had been placed between the new bottom chords ahead of the panel to be changed, were now worked, and the trusses were spread apart a little. The bottom laterals at this point had also been disconnected to allow the trusses to spread. The falls from the traveler boom were tightened, and the floor-beam, *b*, was hoisted to its permanent position. Men with connecting bars guided the floor-beam over the filler plates so that it would not jam. As the new floor-beam was raised, the hangers came up in place under the old stringers in the panel, *bc*. After the floor-beam was up to the proper level, one end was connected to the post by a few bolts, and the falls were taken off that end. The other end was pulled about 3 in. out of line, being held up by the falls. The two new stringers which had been hung on the inside of the trusses, were put in place by the set of falls which had been removed from one end of the floor-beam. These stringers went into place easily, because the floor-beam had been pulled to one side. The floor-beam was then pulled back and connected up at each end with sufficient bolts and drift-pins to carry trains. The falls holding the old stringers in Panel *bc* were slacked off to allow the weight to come on the hangers. New ties, which had been brought out during the operation, were put in place on the new center stringers and the rails were hauled back and reconnected. As a rule, there was some difficulty in getting the rails in on account of the track creeping. Trains were then allowed to pass.

The other two stringers were taken from the material track and put in place in the panel, and the stringer laterals put in. It was difficult to get these stringers in because the fit was so close between their ends and the floor-beams. The bottom laterals were connected up to the lateral plates. The old material taken out during the operation was loaded on cars, and preparations were started for the change of the next panel. The average time that traffic was blocked during this operation was about $1\frac{1}{2}$ hours, the minimum time being 55 min. It took a total of 7 weeks to change the whole floor, but this would have been cut down to 5 weeks, if the north traveler had not been delayed on account of the fire.

After the whole floor had been changed, ties were laid on the up-stream line of stringers, these ties being fitted in between those already laid on the center stringers. Track was laid, and the traffic was turned over to the up-stream track. The ties in the center were taken out and put on the down-stream line of stringers, enough extra ties being added to obtain the correct spacing. The center track was then thrown over, making a double track.

A 30-ton, steel derrick with an 80-ft. boom was erected on Span 1 and removed the two south travelers; then it was erected on Span 5 and removed the two north travelers.

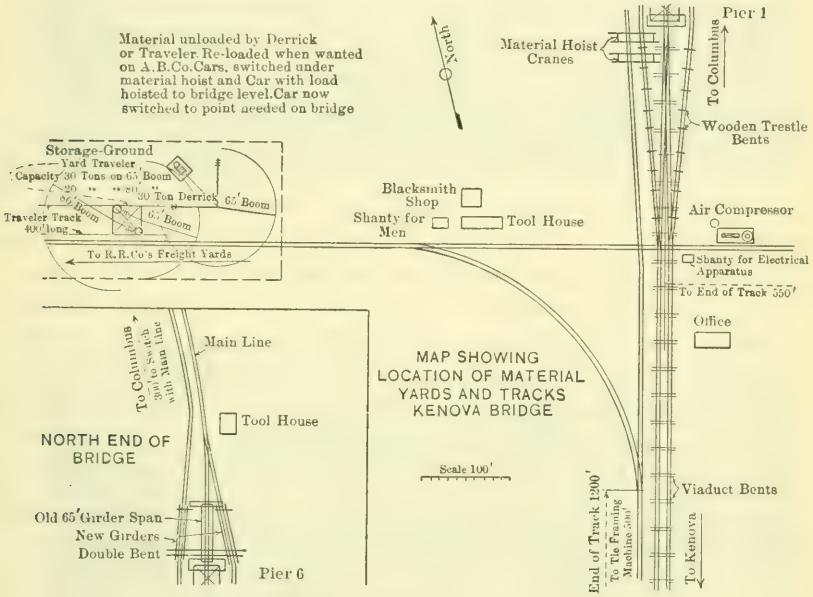


FIG. 31.

ARRANGEMENT OF MATERIAL YARD, TRACKS, ETC.

The layout of the yard, material hoist, and tracks is shown in Fig. 31. The yard was located down stream at the Kenova end of the bridge. It was equipped with one 30-ton derrick with a 65-ft. boom, the derrick being worked by an 85-h.p. steam engine. In addition to the derrick there was a steel traveler with an **A**-frame at each end having a 60-ft. boom on one end and an 80-ft. boom on the other. These booms were good for 30 and 20 tons, respectively. A track

for the traveler was laid on the ground, and this traveler handled all the lighter material. The chords for the 298-ft. spans were unloaded by the derrick; the 518-ft. span chords, each of which weighed more than 40 tons, were unloaded by using the derrick at one end and the traveler at the other. This traveler had previously been used in the erection of the Queen and Crescent Bridge at High Bridge, Ky. It was worked with two, 35-h.p., steam engines placed on its deck. The pier girders were unloaded by skidding them off cars and letting them rest on blocking.

There were seven flat cars on the job belonging to the American Bridge Company, and material was loaded on these and shifted around on tracks under the material hoist, where it was hoisted to the tracks above. On these cars there were attachments whereby the entire car with its load could be lifted and placed on the tracks 50 ft. above. The long pieces, which had to be loaded on two or more cars, were hoisted above and placed on push-cars on top. These push-cars were made from heavy railroad car trucks. The material hoist consisted of wooden bents, across which were placed two steel girders. These girders spanned the track on the ground and also one material track on top. On each pair of girders was placed a steel carriage running on wheels, and a double-drum, electric, hoisting engine was fastened to this carriage. A set of wire-rope falls was attached to each carriage and did the hoisting. A line from the other drum of the engine pulled the carriage across the girders, and a counterweight was used to pull it back again. All the material for the bridge was hoisted to the bridge level by this material hoist, except the end posts for Span 3 and the pier girders. These end posts were sent around to the main line.

One dinkey locomotive was kept in the material yard to do the shifting, and a lighter one was used on the bridge. The material tracks on the bridge had to be connected at each end with the main line. At the south end, this was done by a timber trestle; at the north end, the new 65-ft., deck, plate-girder span was used. A 30-ton, all-steel derrick-car with a 50-ft. boom, was kept on the job to move the gallows-frame from pier to pier and to handle the bottom of the end posts of Spans 1 and 5. After this work was completed, the derrick-car was sent away. A small wooden derrick-car of about 10 tons capacity was also kept on the site to handle light material, timber, etc.

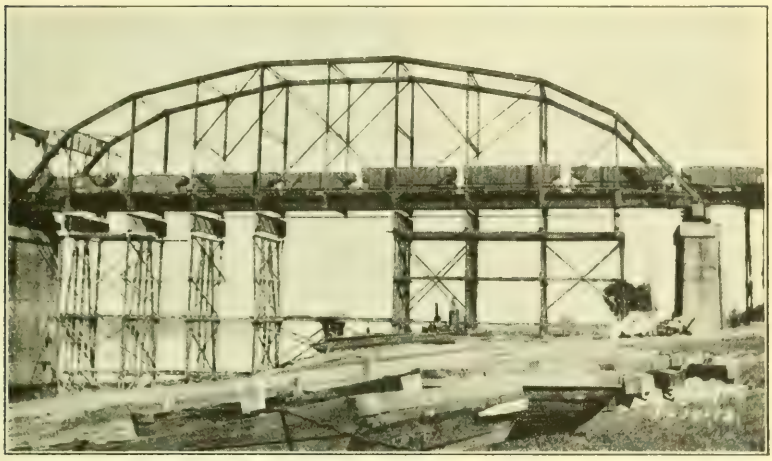


FIG. 32.—FALSEWORK UNDER SPAN 1.

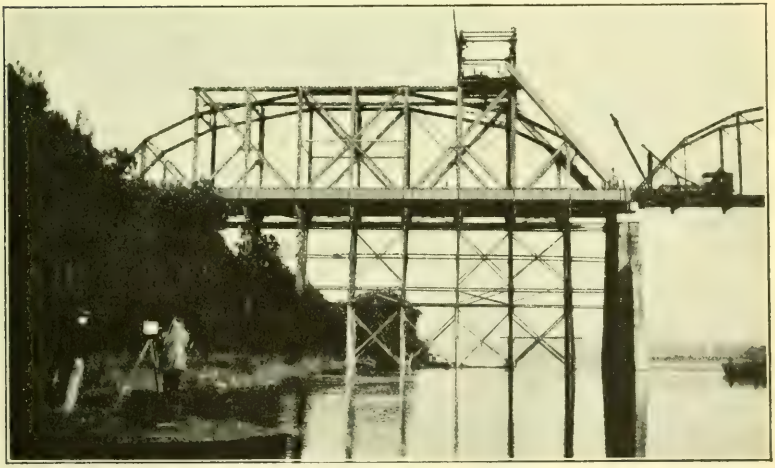


FIG. 33.—PLACING END POST OF SPAN 1.

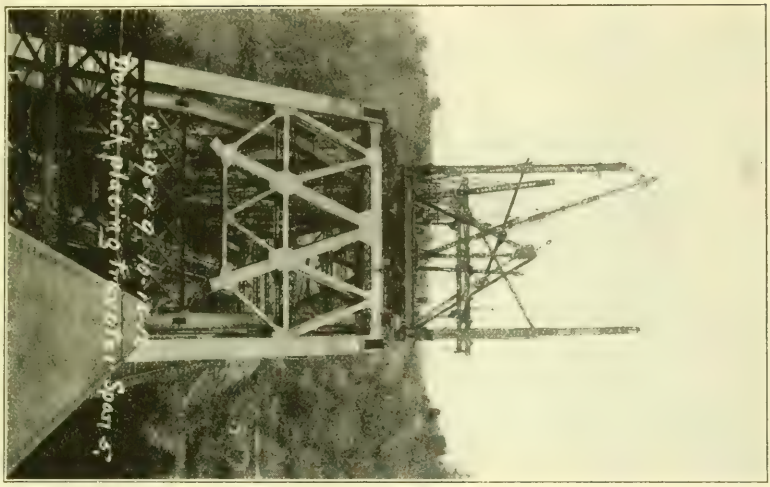


FIG. 34.—PORTALS OF OLD AND NEW SPANS 1.

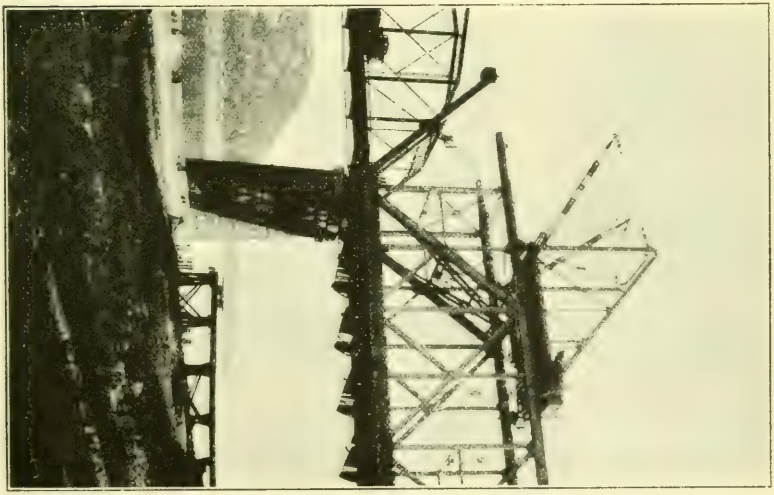


FIG. 35.—PLACING SHORE END POST OF SPAN 2.

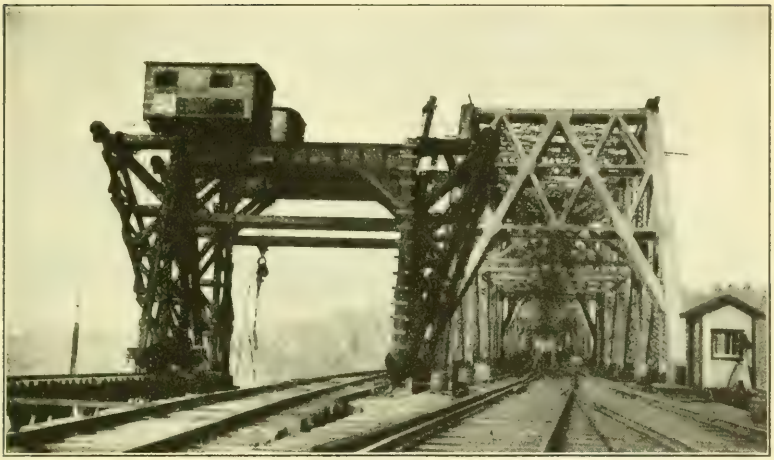


FIG. 36.—MATERIAL HOIST, AND NEW PORTAL.

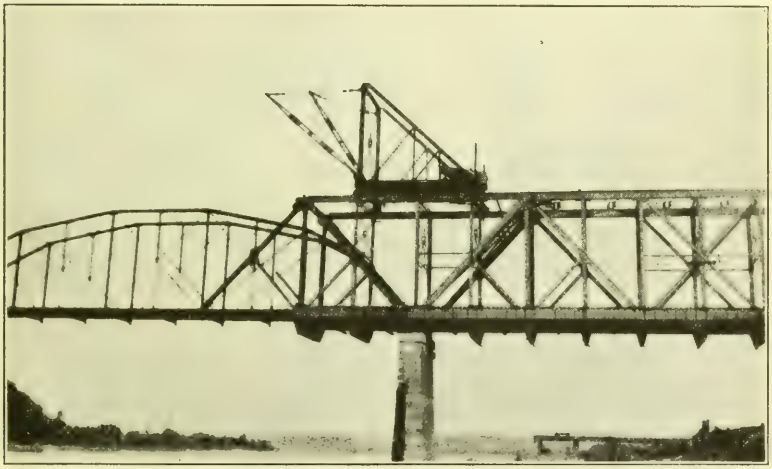


FIG. 37.—PLACING DIAGONAL OF SPAN 2.

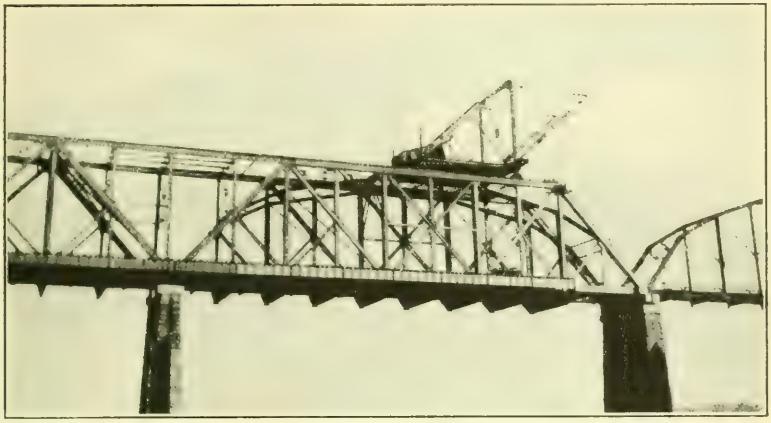


FIG. 38.—PLACING RIVER END POST OF SPAN 2.

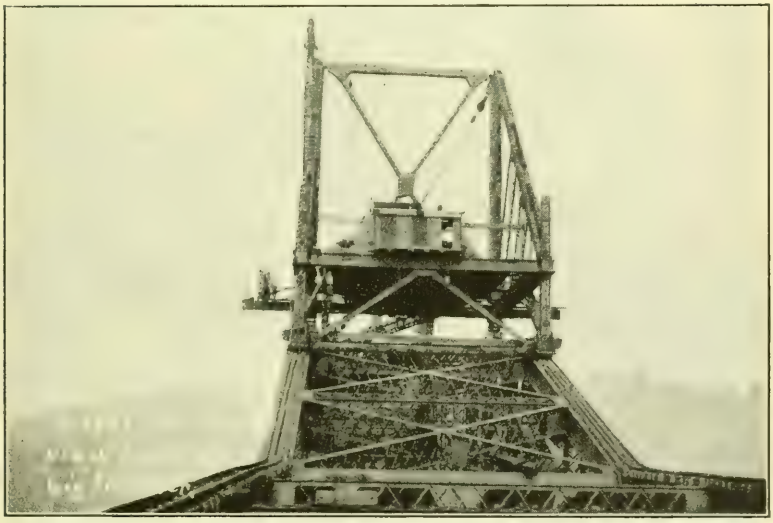


FIG. 39.—LARGE TRAVELER CLIMBING SPAN 3.

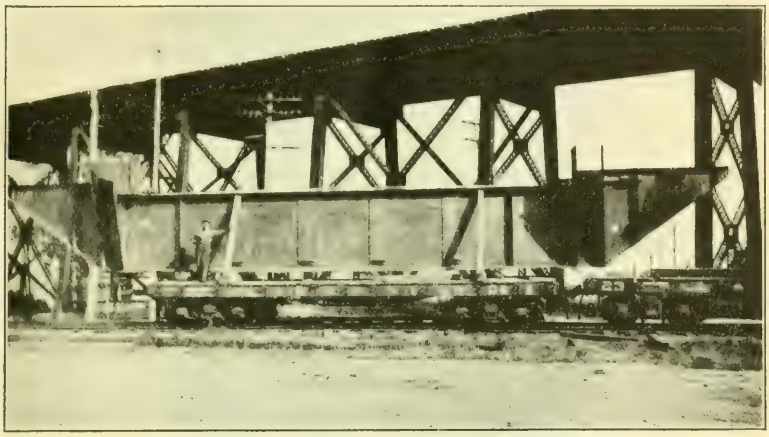


FIG. 40.—NEW FLOOR-BEAM AND TEMPORARY BRACKETS.

FIG. 41.—LIFTING NEW FLOOR-BEAMS INTO PLACE.

FIG. 42.—OLD AND NEW SPANS 3.

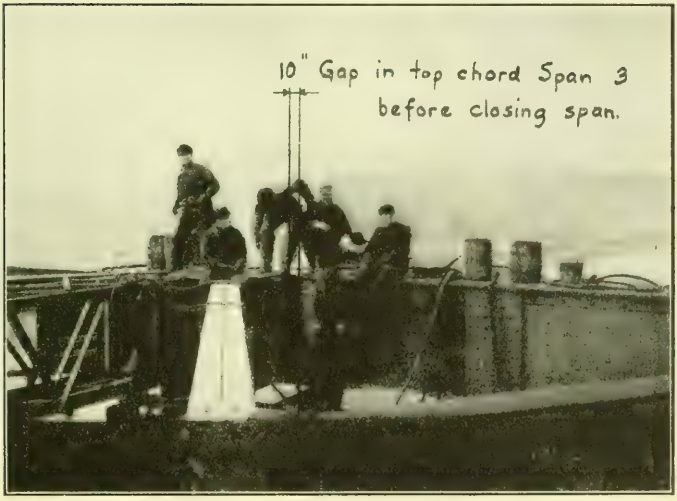


FIG. 43.—GAP AT TOP CHORD, SPAN 3.

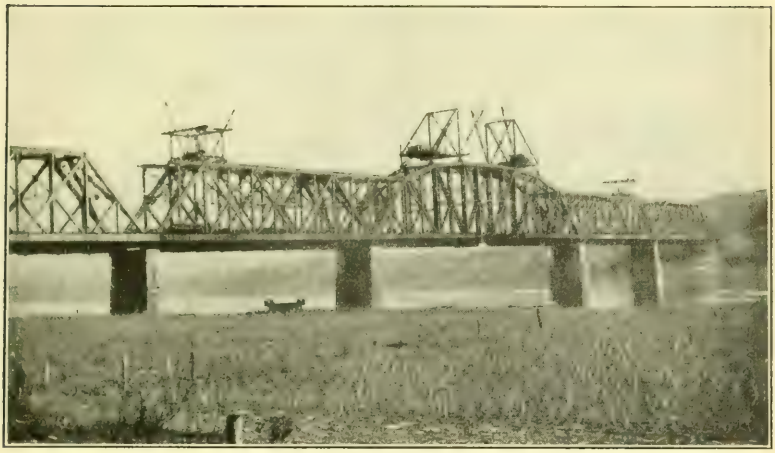


FIG. 44.—SPAN 3, BEFORE CLOSING.

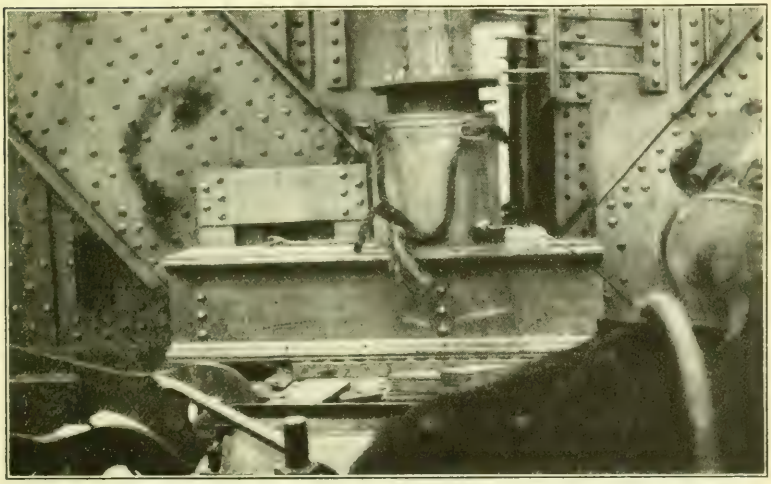


FIG. 45.—JACK, ON ROLLERS, PIER 5.

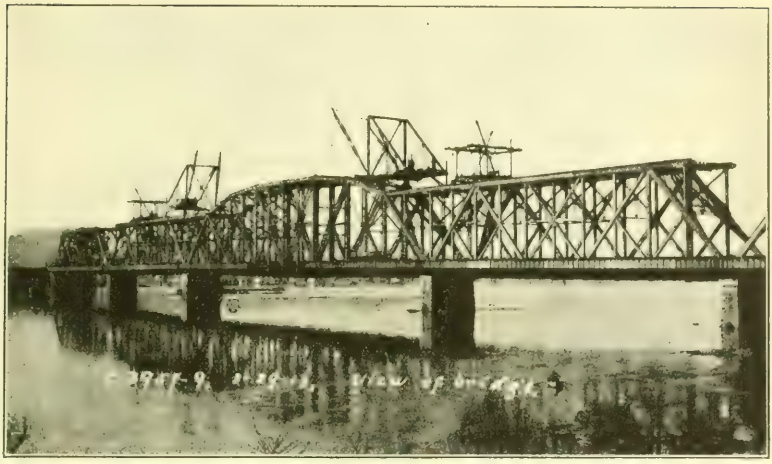


FIG. 46.—KENOVA BRIDGE, AT TIME OF HIGH WATER.

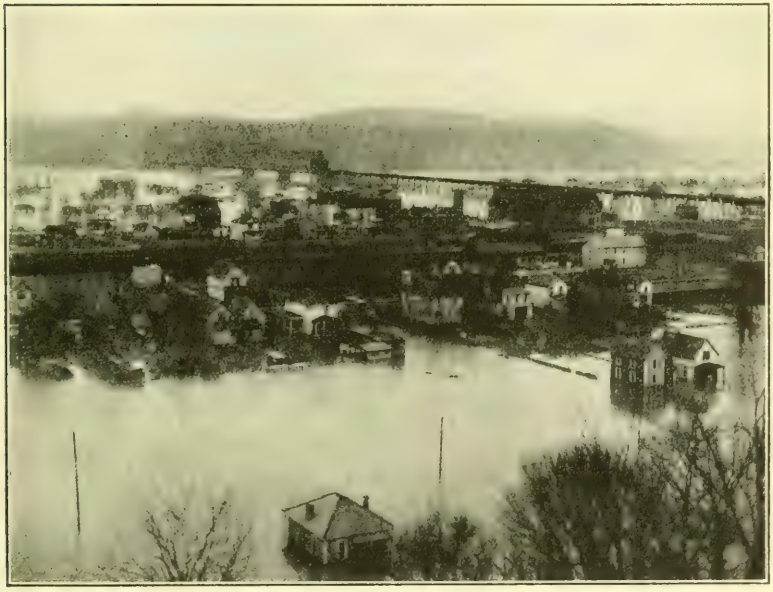


FIG. 47.—KENOVA, W. VA., DURING FLOOD.

There were four electric hoisting engines, two 100-h.p., 4-drum, 8-spool, and two 65-h.p., 2-drum, 4-spool. The two larger engines were used on the large travelers and the two smaller ones on the material hoist. There were also about eight steam engines at different parts of the site. Direct current, at 550 volts, was obtained from a street-car circuit which ran close to the south end of the bridge. The electric engines were found to be much superior to steam for use on the travelers, as there was no vibration from them when on the cantilever arm, the power was always there, and the trouble with coal and water was avoided. Rivets were driven by compressed air at about 100-lb. pressure furnished by a 12 by 12½ by 14-in., straight-line compressor with a capacity of 285 cu. ft. of free air per minute. This compressor was on the ground at the south end of the bridge.

The ties were framed by a tie-dapping machine run by a gasoline engine. This machine had a circular saw for squaring the ends of the ties, and on another shaft there were a number of small coarse-toothed saws to cut the dap. These saws were adjustable, so that a dap of any width could be cut, and there were levers to control the ties so that they would stop in the right positions for the various cuts. The ties were piled in large stacks, and the machine, mounted on four wheels, was moved from stack to stack. The ties were taken in at the front end, framed, and then piled back of the machine. It was found that by this method the framing cost much less than by hand. In removing the old spans, the oxy-acetylene torch was used to burn through the members, and this facilitated the work to a considerable degree.

Conclusion.

The new bridge is shown in Fig. 2. The weight of the approach viaduct is about 2 200 tons, and the weight of the finished river spans is about 9 600 tons. There were about 1 300 tons of erection material, such as the ties between the spans, the material track brackets, etc., making the total weight of steel about 13 100 tons. The entire reconstruction, covering a period of about 2 years, was carried on independently, first, of the heavy traffic over the bridge; and second, of the stage of the Ohio River. There were several floods during this period, the last being the great flood of March and April, 1913, which occurred about a month after the new Span 3 was swung. The water level rose about 65 ft., and the river practically covered the

Town of Kenova, but no damage was done which interfered with the railway traffic.

The general method of the reconstruction of this bridge was proposed by C. G. E. Larsson, M. Am. Soc. C. E., of the American Bridge Company. It was worked up by F. P. Witmer, M. Am. Soc. C. E., now Structural Engineer of the Brooklyn Rapid Transit Company and Mr. William G. Grove, Engineers on designs, under the direction of C. W. Bryan, M. Am. Soc. C. E., Chief Engineer, and Mr. Larsson, Assistant Chief Engineer, of the American Bridge Company. The steelwork was fabricated at the Ambridge Plant. The erection was under the direction of Mr. J. B. Gemberling, Division Erection Manager. The work in the field was in charge of Mr. D. Burns, Foreman, and Henry Taylor, Assoc. M. Am. Soc. C. E., was the Field Engineer. For the Norfolk and Western Railway, the work was under the supervision of C. S. Churchill, M. Am. Soc. C. E., formerly Chief Engineer, and C. C. Wentworth, M. Am. Soc. C. E., Assistant Engineer. J. E. Crawford, M. Am. Soc. C. E., now Chief Engineer, was in charge of the alterations to the masonry piers and the checking of the steel superstructure, and F. P. Turner, Assoc. M. Am. Soc. C. E., now Bridge Engineer, was the Engineer in the field.

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REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

Discussion.*

By Harrison S. Taft, Esq.†

Harrison S. Taft,‡ Esq. (by letter).—The discussions on this paper may be divided into two classes; namely, those by men who have made a thorough study and investigation of the subject, covered by visits to foreign ports; and by men whose knowledge and experience has been confined to the few years during which concrete docks have been building in the United States. Mr. Taft.

The writer regrets that he has not been able to include in the discussion one or two communications he has received from foreign dock engineers, giving the results of their experience with concrete when subjected to sea water action, because in at least one instance, these results have been radically different from those at some American ports. Whereas, what few failures have taken place in the use of concrete in sea water structures in America seem to have been between tides, it appears that at Southampton, England, "below high water of neap tides the reinforced concrete is absolutely perfect, except in a few places where evidently it has been damaged by blows * * * the rusting action all taking place above high tide level". The writer's informant states that electrolysis has been the cause of this rusting, and that, in building concrete docks, it is of vast importance that "the electric mains should be carefully insulated so that the current cannot pass through the reinforcement".

* Continued from December, 1914, *Proceedings*.

† Author's closure.

‡ Seattle, Wash.

Mr. Taft.

As for any attempt at expanding the contents of the paper, the writer feels that the ground, in general, was covered so thoroughly that no further discussion is needed at present. He admits that the whole subject of the use of cement in sea water structures is one in the actual handling of which a large amount of knowledge has yet to be acquired by concrete engineers, and an equal amount of training given to concrete foremen and their crews, before they can become past masters in the art. For, from what does the foreign engineer get his confidence in concrete structures exposed to sea water conditions, but from actual experience and past successes. On the other hand, whence comes the skepticism of American engineers on the same subject, but from lack of thorough knowledge of the art and failures resulting therefrom.

Mr. Stern seems to feel that the writer did not dwell sufficiently on the failure side of the question, and overlooked a case, which he cites, in the lower part of New York City. It appears to the writer that in Part III of the paper sufficient note was made of a number of bad failures on both the Atlantic and Pacific Coasts to show their extent, without citing each and every case that ever happened. The writer most certainly did not try to bring out the successful uses of concrete subject to sea water action more prominently than the other side of the discussion; but endeavored to treat the subject matter of Part III on an impartial basis, and regrets that answers were not forthcoming in explanation of some of the failures in the use of concrete mentioned therein.

As Mr. Stern pointed out, cement chemists and manufacturers must give the concrete engineer a cement suitable for sea water structures, if the art of building concrete docks is to progress; but it is of equal importance to have a suitable sand, gravel, or stone, and of still more importance that the contractors, shall handle the material in such a way as to obtain an impermeable concrete, because, in the final analysis, and as Mr. Coombs has brought out, pre-moulded concrete, of a rich mixture, made of suitable materials is the prime requisite for the most successful use of concrete when subjected to sea water action. In no case should the salt water come in contact with green concrete.

In stating that in using pre-moulded concrete the jointing together of the different members of the structure into a monolithic mass is a difficult accomplishment, and that the existence of joints below the water is of a serious nature, Mr. Walker utters a truism. There is no doubt that a true monolithic structure is desirable, if such a mode of construction can be adopted economically. On the other hand, has the pre-moulded system reached its final and ultimate development, and do not pre-moulded concrete piles with their heads embedded in the deck members, the latter poured *en masse in situ*, offer

a remedy for the objectionable features, mentioned by Mr. Walker, in such a system of construction? Mr. Taft.

The writer doubts very much that the art of concrete construction under sea water conditions has yet reached its ultimate development. In fact, he knows it has not, or else concrete engineers would be at a standstill, and such a condition is absolutely contrary to American engineering. During the past year or so the writer has been working on this very problem in order to overcome the objectionable features in the pre-moulded system, which he recognized some time ago, namely, the joints in concrete exposed to sea water action where the steel reinforcement passes through the joint. If such a thing can be obliterated—and it is possible to do so—does not such an objection to a pre-moulded structure become a negative quantity?

In reviewing the criticism by Mr. Davis, it would appear that he and the writer had gone over the same road, in their study and investigation of the subject, as there seems to be hardly a thought in Mr. Davis' most comprehensive statements which does not coincide with the writer's ideas and with the ideas of a large number of leading dock engineers of foreign countries. Mr. Davis' discussion is full of most valuable suggestions, relating to the construction side of the problem, as well as the actual handling of the concrete material of dock work, and can be studied with great advantage by all engineers engaged in such work.

Through all the discussions the writer has noted references to questions as to the commercial life of temporary *versus* permanent structures. As Mr. Barney points out, and as stated previously by the writer,* the one great question in all such discussions is that of the "financial and commercial life"—a vital factor only too often ignored. At some future time the writer trusts that he will have the privilege of laying before this Society the results of his studies of dock finance, wherein, with charts already prepared, the relative position of permanent and temporary dock structures is clearly portrayed. Briefly stated, the problem is this:

If, during the life of a wooden structure, it can earn its depreciation charges, fire insurance, operating expenses, interest on investment (supposedly a bond issue, as with most public improvements), and pay off the bonds, then perhaps such a structure is of advantage, except with reference to the general fire hazards to surrounding property; but, if the wooden structure cannot do this, and the bond issue is for a longer period than the life of the dock, a permanent structure is more advantageous than a temporary one.

In closing, the writer desires to thank the members of the Society for the courtesy shown him in the generous treatment of his paper.

* In his discussion of the paper by Charles W. Staniford, M. Am. Soc. C. E., entitled "Modern Pier Construction in New York Harbor," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 525.

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THE CONSTRUCTION OF THE KLONDIKE PIPE LINE.

Discussion.*

By W. W. Edwards, Assoc. M. Am. Soc. C. E.†

Mr. Edwards.

W. W. Edwards,‡ Assoc. M. Am. Soc. C. E. (by letter).—The writer urges that no attempt be made to build a structure, or to grade, on frozen ground, if there is any practicable way to avoid it. In the case of the Klondike pipe line nothing else could be done. The same condition has been observed in many other instances. Building on "frost" is at best a makeshift, and one never has the feeling that he has done a good job. A structure founded on frozen ground will probably need jacking up into shape once in a while, and a railroad over frozen ground will need considerable work by the maintenance-of-way gang with tamping and line bars.

Probably the most trying work which the writer has ever observed is ditching in frozen ground. When this is attempted, the right of way should be cleared of trees, brush, and moss, at least a year before the excavation is to be made, so that the sun's heat will thaw the ground to the required depth. In ground which is not too steep or does not contain too many ice stringers, this method may result in a good ditch; but where the ground surface has a considerable slope, the ditch excavation generally starts slides from above, which cause much trouble.

Then, again, the finished ditch may appear to be all right, but, when water is turned in, it will percolate until it reaches an ice stringer along which it follows until it comes out to the ground

* Continued from September, 1914, *Proceedings*.
† Author's closure.
‡ San José, Cal.

Mr. Edwards.

surface below. In this way a ditch may break in the bottom, and the water may come out to the surface 100 ft. or more below the grade, leaving the ditch bank intact. Repairing such breaks requires brush, moss, soil, and much skill and patience in puddling. Ditches in such material are generally made wide and shallow, the lower bank being protected on the inside with strips of moss.

The writer does not wish to give the impression that all the soil in Alaska and Yukon Territory is frozen the year round; but spots of frozen ground are frequently encountered, especially in the Yukon Basin, and, as in the case of the Klondike pipe line, often cannot be avoided economically.

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THE CONSTANT-ANGLE ARCH DAM.

Discussion.*

By Lars R. Jorgensen, M. Am. Soc. C. E.†

Mr. Jorgensen.

Lars R. Jorgensen,‡ M. Am. Soc. C. E. (by letter).—It is possible in most cases, to dimension the length of the up-stream radii, on a constant-angle arch in a **V**-shaped canyon, in such a manner that the crown will deflect in a straight line, this deflection being a maximum toward the crest and nearly zero at the bottom.

If this cannot be done, the crown will deflect nevertheless practically in a straight line by transferring stresses from one elevation to another through shear and bending. It is also very likely that some transference is done, as suggested by Mr. Henny, in Figs. 18 and 19, in all cases. The customary allowable arch pressure, varying between, say, 15 and 25 tons per sq. ft., is sufficiently low to take care of some local concentration of stress.

Although French engineers have been responsible for much of the development work in dam design, statements made by some of them, as to the relation which ought to exist between thickness of dam and length of up-stream radius at the bottom, can only be taken as expressions of their personal opinions.

As to the question whether or not a gravity section ought to be used for the arch, no general answer can be given. Where the project will stand the expense of large safety factors, the dam, of course, should be designed with at least the same factor of safety as that used on the other structures of the same project. This, perhaps, necessitates the use of low arch stresses, and, consequently, a heavy section is the result, but the material should be placed to the advantage of the arch, as this form of structure is the most economical for closing a reasonably short and deep gap.

As to the sentence with which Mr. Wegmann takes issue, the writer begs to state that in certain cases it might be possible that the

* Continued from December, 1914, *Proceedings*.

† Author's closure.

‡ San Francisco, Cal.

Mr. Jorgensen.

actual difference in unit compression between the up- and down-stream faces will be the same for an arch and a gravity dam, but, if calculated on a percentage basis, the distribution of stresses in the arch will be the more even of the two. For instance, an arch calculated for an average stress of 25 tons per sq. ft. may have this stress distributed in the ratio of 30 tons at the down-stream face and 20 tons along the up-stream face near the abutments, due to the curved beam action and the great thickness of the arch. If this distribution is compared with that in the gravity section having 10 tons unit compression along the down-stream face and zero tons unit compression along the up-stream face, the actual difference would be the same in each case, but, calculated on a percentage basis, the distribution of the arch stresses is seen to be much the better.

In the middle of an arch dam the curved beam action tends to neutralize whatever difference there would be otherwise between the unit compression along the two faces, if the proportions of structure and site are such that this action takes place to a considerable extent. The difference between the maximum and the minimum arch compression in the middle of the dam (center of canyon), therefore, is small, although toward the abutments it may be quite large. Toward the abutments, however, there is little or no movement of the dam, and, therefore, higher local compressive stresses can be tolerated than toward the middle portion of the dam, which moves back and forth with the rise and fall of water level and temperature.

To Mr. Brodie's question as to whether the method given for finding the proportion of the load taken by the arch and by the cantilever, respectively, is applicable to his spillway arch dam, the writer can only state that it is applicable for arch dams having deflection curves similar to Curve *A* of Fig. 9, and, in order to have an arch deflect thus, it is necessary that its up-stream radii decrease in length from the crest toward the foundation; this is what makes a better arch.

The formula in Mr. Shirreff's discussion, referred to in the paper, is correct, but the writer preferred to use Mr. Shirreff's expression (9*b*) as the type in the final formula did not seem plain. Mr. Brodie's point, as to whether the full water pressure, *H*, in Formulas (6) and (7) should be taken, or only a percentage, when considering the influence of Poisson's ratio, is well taken.

It is true that the water pressure on the up-stream face is not opposed directly by any opposite force, but it is opposed, however, by two reactions directed through the two abutments; therefore the full head, *H*, of water has been substituted in Formula (6), as it has also been used by modern French engineers. When working with concrete, Poisson's ratio cannot be regarded as an exact fixed quantity, $\frac{1}{5}$; therefore, a smaller error in the determination of *H* in this matter cannot be regarded as very serious.

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THE CLARIFICATION OF SEWAGE BY FINE SCREENS.

Discussion.*

By Messrs. George T. Hammond, J. C. Riedel, David T. Pitkethly, and William L. D'Olier.

Mr. Hammond.

George T. Hammond,† M. Am. Soc. C. E. (by letter).—This paper, on a timely subject, is of unusual interest in that it gives, in condensed form, a very complete treatment of the subject. The writer believes that the fine screen has a large field of usefulness, both as the sole method of sewage treatment under certain conditions, and also as an essential part of methods which aim to secure a higher degree of purification. While making studies and investigations, preparatory to the design of an experimental sewage disposal plant for the Borough of Brooklyn, the writer accompanied Rudolph Hering and E. J. Fort, Members, Am. Soc. C. E., through England, and Mr. Fort through Germany, for the purpose of observing and noting the state of the art of sewage treatment, with especial reference to screens, filters, tanks, and the various structures and appurtenances used, and was impressed with the high efficiency of the screening plants, especially in Germany, and the great extent to which they have come into use there as an entire method of sewage treatment.

Although there are a number of fine screens in use in England, they generally form a part of a more extensive treatment, and their purpose is preparatory to other measures. The English rivers are small, and the population crowded together. Several large cities elbow each other on the same small stream which, from its source to its mouth, has to carry away the effluents of a series of disposal plants, located at short distances from each other.

* Continued from December, 1914, *Proceedings*.

† Brooklyn, N. Y.

Mr. Hammond.

In some places, as at Birmingham, the sewage-plant effluent is much greater in volume than that of the stream into which it is discharged. Such conditions require the highest grade of treatment, regardless of expense, and the most highly developed disposal plants in the world are the result. In Germany, the rivers are larger and usually have a swiftly flowing current, thus making disposal by dilution possible in many places after the removal of the grosser materials by screens. In England the settling tank, as a rule, is used as the essential reliance of every disposal plant, and some method of filtration is provided. In Germany the fine screen is making rapid progress as a complete method, a single type of screen having been introduced in more than fifty cities, not to mention many other types which have been used extensively. Even where tanks are used, such, for instance, as the Emscher tank, it is generally the purpose to remove, as quickly as possible, matters which will settle, and not provide long periods of septic action; such tanks, in fact, perform the service of screens to a great extent, and do it with a high degree of efficiency. It might, perhaps, be successfully maintained that such tanks are water-screens, which possess the added advantage of being able to digest the screenings. German practice in sewage disposal, guided by scientific methods of study and management, aims at securing a satisfactory result, and does not justify the expenditure of public funds for obtaining a higher degree of sewage purification than this demands.

There are only a few places in England where fine screens would give an effluent of sufficient purity to prevent nuisance, but, in Germany, cities like Hamburg, Dresden, Bremen, and many others, are sufficiently well served with screens alone; and, where tanks are used, as throughout the Emscher District, the period of sedimentation is short, and the effluent is comparable to screened sewage.

In Hamburg, through the kindness of Dr. Dunbar, Mr. Fort and the writer were given every opportunity to make observations on the operation of the screening plant and the condition of the Elbe River. As this screen is described by Mr. Allen, further comments are not necessary. It can scarcely be called a fine screen, in the sense that the Dresden screens are fine screens, but careful observations failed to reveal any floating material from the sewage in the river—and these observations, extending over several days in June, were made from Dr. Dunbar's launch and also from other vessels moving on the river.

The condition of the river, from the water-works intake for Altona, below Blankenese, several miles below Hamburg, to a point several miles above Hamburg, was examined and found to be good, with no visible evidence of pollution at any stage of the tide, Dr. Dunbar stating that it was the ordinary condition. From a boat on the surface it was so difficult to find the points at which the sewage

Mr. Hammond.

enters the river that Dr. Dunbar was not sure that he had located them correctly until, after careful observation of landmarks, he was able to state that the boat was directly over the area affected by the discharge, and the writer would not have known it had it not been for this assurance. In as busy a harbor as that of Hamburg, in spite of all regulations to prevent pollution, it would not be possible to prevent the occasional dropping into the water from shipping of some floating substances derived from food under preparation, or cast away —the usual wastes incident to shipping and traffic. Such materials would be present if no sewage entered the river, and the writer marveled that so little matter of this nature was to be seen. The presence of sea-gulls has been cited by some observers to prove the presence of pollution, but, although the writer observed a few, he carefully noted that they did not seem to find any food, and he did not observe any of them near the outlets of the sewers or near the screening plant.

This plant was nearly perfect in operation; it was so clean, in fact, that the engineer who took Mr. Fort and the writer through it and explained it in detail, was accompanied by his daughter, a young lady, who aided him as interpreter.

The screens worked perfectly, operating and cleaning themselves automatically; and all handling was done by machinery. The screenings were kept under cover, from the plant to the covered scow in which they are removed, and no odor was noticeable.

While investigating the river, a number of side basins were visited, as they seemed to be places where floating matter might tend to collect and remain, but there was neither floating matter of consequence nor visible evidence of pollution. When one looks into the river, the water, as in nearly all very large rivers flowing through extensive alluvial plains, has a slight brownish-green color, but samples taken in glass show no trace of this color; a moderate quantity of green algæ growth shows on piles and bulkheads. It was stated by Dr. Dunbar and others that the oxygen content is always satisfactory. The volume of flow is large, the tidal range being between 6 and 7 ft., but the water is always fresh, and, after filtration, is used for the water supply of Hamburg and Altona.

Mr. Fort and the writer visited the Altona Water-Works, about 10 miles down stream from Hamburg, as the guests of an Altona official who had passed much of his life in the United States, and most of the day was spent in examining all parts of the works, then being enlarged and improved. The water is admitted, during high tide, from the river to a series of large settling tanks at the river level; from these, after precipitation, it is pumped to the top of the hill or bluff, 280 ft., and filtered. The filtration plant, originally installed and described,* many years ago, by the late James P. Kirkwood, Past-

*"Report on the Filtration of River Waters", p. 120 (1869).

Mr. Hammond.

President, Am. Soc. C. E., has been reconstructed, the Jewel type of filters having been recently introduced. The large settling basins along the shore gave excellent opportunity for observing the average condition of the river which, 10 miles up stream, receives the sewage of approximately 1 200 000 people; and it must be said that it was satisfactory, there being no apparent signs of pollution.

After examining a considerable number of German sewage screening plants, the writer formed the opinion, which has been strengthened by reading Mr. Allen's paper, that a more extensive use of such plants in the United States, at small cost, would go very far toward the protection of our rivers from pollution, and under many existing conditions give all the necessary protection. Where the volume of water into which sewage is discharged is sufficient to take care of the finely divided impurities remaining after screening, no further treatment is required to prevent a nuisance, provided the screening plants are properly designed and carefully operated.

There remains the important problem of getting rid of the screenings. The writer was much impressed by this fact at Dresden, where the Riensch-Wurl screen, described so admirably by the author, seems to meet every requirement in sufficiently treating the entire flow of sewage, but where a pile of screenings, about 100 ft. long, 6 ft. wide and 6 ft. high, awaited removal, and gave forth no very pleasing odor.

The disposal of screenings is a subject of great importance, and the writer would be thankful for more extensive data on this aspect of the matter.

The removal from sewage of matters which settle, if this can be effected before septic action has intervened, seems to be indicated both by experimental experience and theory. Sewage, as it reaches the outlet of a well designed system, is ordinarily in a fresh state; if, under these circumstances, the organic matters capable of removal by a fine screen are taken away at once, their liquefaction in the sewage is prevented, and the ultimate demand for oxygen is greatly decreased. The sewage thus screened, while fresh, may have suffered some increase in colloidal matters in emulsified form, but this is of slight consequence in comparison with the benefit arising from the removal of such large masses of putrescible matter as these screens secure. Thus prepared, the screen-effluent may be given further treatment, as the situation demands, by percolating filters or contact beds, or, if the river flow is sufficient, it may be discharged through multiple outlets into the selected waterway; if it is desirable to protect shell-fish, it may be disinfected, and will require less disinfectant than a tank effluent which has had time to become septic.

It must be remembered: that the operation of a screen is positive, and must be accomplished with acting parts and a motive power; it will cease to operate if not carefully managed and kept in order; and

it does not dispose of the screenings, but rather gives rise to a new but smaller problem than the one it is intended to solve. It solves a large problem, but creates the smaller one relative to the disposal of the screenings. Mr. Hammond.

In comparison with this, a well designed Emscher tank is quite as efficient as a screen in the removal of matters which settle and will take out as great a quantity, in parts per million, in from 20 to 30 min. retention, and do it in such a manner that the effluent will be in a better condition for subsequent treatment than that from the screen; and, furthermore, there need be no screenings to dispose of, as sufficient storage capacity can be provided in the digestion chamber to take all settleable and screenable matters and hold them without nuisance for 6 months at a time. Moreover, this tank does not require the high priced attention and care which must constantly be given to the screen and its operation, and when the digested materials are removed, they have been reduced to a small fraction of their original volume, and are incapable of causing a nuisance by bad odors. The Emscher tank is considered undoubtedly the greatest advance in sewage treatment in recent years. However, it is not without some difficulties, high cost of installation being the first; and frequently its operation is possible only if the sewage is pumped, which adds a positively operated unit to this method and increases the cost greatly; and although it will operate with a minimum of care, it must have that care regularly and constantly. Sludge drying beds must be supplied, or the sludge will have to be removed wet in portable tanks; but it is thought that this will not prove as great an expense as the removal of fresh screenings from a screen plant; the latter must be removed in a similar manner, in most cases, and are far greater in volume.

The selection of a screen plant or a tank plant for a given project must be determined by the conditions and the requirements of the case; and the sanitarian must look at the matter from a number of angles before he decides which to use.

It seems to the writer that the fine screen plant is indicated for extensive use in great seaboard cities, and in cities on large bodies of water, in which screening will prove sufficient for local conditions, and will give complete satisfaction. These screens will also be used where sewage farming in any of its forms is adopted, and in connection with sewage disposal plants for the purpose of removing scum-forming material from the sewage previous to tankage and also for protecting sprinkling filter nozzles from clogging.

J. C. Riedel,* M. Am. Soc. C. E. (by letter).—Mr. Allen's paper will unquestionably become a classic on this subject. The data presented will furnish the basis for comparison with experiments, Mr. Riedel.

* Brooklyn, N. Y.

Mr. Riedel. now being made and hereafter to be undertaken, in those cities and towns where fine screens may be used advantageously.

The time is rapidly approaching, if it is not already here, when cities and towns bordering on tidal streams will be required to screen all their sewage which, in many localities, is turned, untreated and unscreened, directly into the streams. One matter of interest to the writer, which receives slight mention in the paper, is the loss of head through the screens. It is obvious that this will vary with screens of different types, but it is believed that, if any data on the loss of head in any screen are available, many engineers would feel grateful if they were presented. The writer will thank Mr. Allen if, in his closing discussion, he will comment on this subject and present any additional data on the loss of head that he may have available.

Mr. Pitkethly. DAVID T. PITKETHLY,* ASSOC. M. AM. SOC C. E. (by letter).—Mr. Allen is to be congratulated for the admirable way in which his paper is presented, and for its completeness and detail. The paper is a most valuable one to the Engineering Profession. The data presented are in good shape for use and comparison, and the bibliography attached is very thorough.

One feature of screening, on which the data are somewhat lacking, is the loss of head through screens of various types.

After the removal of suspended solids by screens, the most essential part of the problem is their final disposition and the prevention of a nuisance by their accumulation. This is a most important detail in the operation of a screening plant, and more data on this subject would be of value to the Profession.

Where other means of treatment are in use, which is usually the case in cities having the separate system of sewerage, the major portion of the storm flow is generally passed directly into the nearest body of water without treatment, and, as large quantities of suspended solids are carried by storm-water, it would seem that screening this flow would be the proper means of clarifying it and thus rendering it suitable for disposal in water without endangering the condition of the body of water into which it discharges.

In Boston, where rough screening is in operation, and where the course of the sewage, on discharge at ebb tide, can be traced for miles by the floating solids, a finer type of screen could very well be placed, instead of the bar screens now in use.

In many instances, fine screens could be placed where settling and Imhoff tanks necessitate pumping, and where thoroughness of treatment is not essential, thereby reducing the first cost of the plant.

Mr. D'Olier. WILLIAM L. D'OLIER,† ESQ.—This paper is an able presentation of the subject of the development and application of fine screens for the

* Brooklyn, N. Y.

† Philadelphia, Pa.

clarification of sewage. Mr. Allen's very complete data indicate thorough research and study of the subject, and an intimate knowledge of the installation and operation of screens, obtainable only by personal observation of the apparatus in service. He has contributed to our records in English a valuable addition on the subject of sewage treatment. Mr. D'Oher.

That fine-screening is rapidly becoming a recognized method of sewage treatment is shown by the consideration it has received by such eminent authorities as Rudolph Hering, George H. Benzenberg, Edwin A. Fisher, and the late Emil Kuichling, Members, Am. Soc. C. E., in connection with the problem at Rochester, N. Y. The International Joint Commission mentions screening as an acceptable method, and numerous sanitarians, engineers, commissions, and boards of health of various States are approving it as suitable. The Metropolitan Sewerage Commission of New York City and the New York Sewer Plan Commission both endorse screening treatment as acceptable and sufficient for Greater New York.

Fine-screening of sewage, with disinfection of the effluent, is endorsed as a sufficient treatment to afford proper protection to the shell-fish industry.

With just appreciation of the possibilities of dilution, and the self-purifying power of water, or the spontaneous purification of water, acknowledged by authorities, and with due consideration of the vastly increased digestive power of water for screened as compared with raw sewage, as shown in the Thames, England, in the Elbe, Germany, and in home waters, the treatment of sewage by fine screens will be deemed acceptable, and the effluent may be discharged safely into most of the waters of the United States. Numerous coast and river cities offer a very large field for the application of independent screening plants.

The speaker believes that Mr. Allen has not been sufficiently liberal in his statement that the field for fine-screening is distinctly limited. It must be admitted that this has been so, and has been due to a lack of appreciation of its possibilities, but, with the operation and results of tank-treatment plants, better known from actual experience throughout the United States, the Engineering Profession has realized the lack of entire success with, and certain disadvantages in, tank treatment, and, consequently, is more willing to consider other methods, hence the appreciation of the merits and possibilities of fine screens. Therefore, the field is broadening. Not only is the screen accepted as auxiliary apparatus, but, more particularly, as apparatus for independent treatment.

When it is considered that the foul matter in sewage is mostly solids, and that sewage treatment primarily consists of the removal of solids, the sooner they are removed the more complete is the removal.

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If they are removed from sewage while fresh, the pollution of the liquid mass is prevented; that is, the solids, particularly fæcal matter, are not retained to be dissolved and comminuted in time and with travel.

Absolutely fresh sewage may be filtered, producing an effluent clear and stable, thus indicating the importance of early treatment to prevent pollution.

The author's statement that, of the suspended solids removed by screens, two-thirds or more consist of organic matter, may be bettered when fine-screening is accomplished at the proper point and time.

Tanks operate on sedimentable solids; this is particularly true in present-day practice, with the discontinuance of chemical coagulants and precipitants. Therefore, tanks operate only on a percentage of the total solids in suspension, generally from 60 to 75% of the total solids being sedimentable. Results obtained with settling-tank plants in the United States during the past year show a removal of approximately 50% of total solids in suspension.

Although the percentage of removal of solids has been lessened in tank treatment by the discontinuance of chemical treatment and the shortening of the detention period, it must be remembered that fine screens have been improved in design and construction, and that they act, not only as a screen, but as a filter, removing a percentage of non-sedimentable as well as floating solids.

Fine screens may be termed "classifiers," as they insure an effluent which varies only slightly in retained solids. A screen will take out all solids, sedimentable and non-sedimentable, larger than a certain predetermined size, and is not affected by any variation in flow or quantity of solids contained in the sewage. In any tank system the reverse is true; for instance, if a tank will remove 50% of the sedimentable solids, those remaining in the effluent are bound to vary in direct proportion to those in the influent, in addition to the non-sedimentable solids which the tank will not remove; this is also affected by any change in the detention period on account of the varying rate of flow. The condition of the effluent is an important feature.

Having determined the degree of treatment necessary, it should be maintained, regardless of any increase in the volume of solid matter. Screens will deliver a constantly uniform predetermined effluent. It is a difficult matter to ascertain what percentage of removal of non-sedimentable matter is effected by a screen, as the generally accepted method of reading tank results—the cone measuring glass—shows only sedimentable solids. This method has been generally accepted for reading all screen results, hence, in test readings, no credit is received for the removal of non-sedimentable solids.

Mr. Kuichling, who was a student of and advocate of fine screens, has stated that "properly designed and operated fine screens of one

millimeter openings will remove as high a percentage of solids as would the best designed tanks with 1½ hours detention." To appreciate the truth of this statement, it must be borne in mind that the fine solids—those which have been comminuted to extreme fineness—are those which tanks will not remove successfully, and which cause the greatest trouble in the consumption of oxygen, thus affecting the sewage and the water into which it is discharged. Mr. D'Olier.

Further, in order to accept Mr. Kuichling's statement regarding the operation of fine screens, it must be fully determined what the general statements regarding the percentage of removal of solids by tanks really mean. Such general statements as "tanks will remove 90, or 80, or 70%" (at times followed by modifying clauses), may be quite misleading unless considered carefully and intelligently. Such a percentage of removal of some classes of solids may be accomplished, but not of the total solids in suspension. Careful reading of general statements regarding the removal of solids will show that some factor must be used to determine the full percentage of removal of total solids, or, that the tests have been conducted inefficiently or incorrectly.

There is every reason for the author's suggestion that prescribed methods of tests and readings should be established.

The speaker has observed effluents from tank-treatment plants, which could have been improved materially by proper fine-screen treatment.

Plants for fine-screening may be more conveniently located and will require very much less area than those for tank treatment, effecting invariably large savings in the construction of long discharge sewers or interceptors, and of ground area.

With tank-treatment plants it is invariably necessary, or deemed prudent, to purchase extensive land tracts, representing large land costs, in addition to miles of connecting sewers or force mains necessary to reach remote sites.

In addition to the large field available for independent screening plants, a large and extensive field for fine screens is their use as primary or auxiliary devices in extensive treatment. It is generally considered good practice to place ½ or ¼-in. screens on lines leading to tanks, as protection to apparatus, piping, valves, etc., against mechanical obstruction. The advisability of using screens as an aid to tanks is apparent when due consideration is given to the non-digestive nature, or the tendency to resist tank digestion, of various solids which have been generally supposed to submit quickly to digestion. The introduction of such materials into the digestion chamber not only occupies space which has been provided at far greater expense than screens would require, but retards digestion in the entire chamber.

Exhaustive tests and studies of this subject, made during the last several years by Dr. Dunbar, Dr. W. Favre, Dipl. Ing. Spillner,

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and Dr. Guth, have furnished definite data and knowledge of the digestive and non-digestive organic substances, which, it might reasonably be supposed, could be properly deposited in tanks, but which, the results of the experiments would indicate, should be screened out, in aid of tank operation. In many American cities, appreciable quantities of leaves, stems, twigs and match sticks—all vegetable substances which will not readily submit to digestion—tend to burden tank operation and delay digestion. Further, hair and light materials, which tend to form scums, seriously retarding the proper operation of tanks, may all be removed by ¼-in. screens.

The value of fine screens, following tank treatment, to protect sprinkling nozzles, is shown by their successful operation at Baltimore, where the labor necessary to keep the nozzles clear is reduced materially when the screens are in operation.

An application of fine screens which has not yet been adopted, though suggested by several engineers, is that for raw sewage, with finer screens following chemical treatment.

At present, storm-water is discharged without treatment, but this matter has recently been receiving more consideration and endorsement; for such service, fine-screening is particularly well adapted. An important reason for treating storm-water is the fact that it is distributed over a far greater area than the general or dry-weather flow, and thus pollution carried by it goes beyond the limits or confines in which it may be feared or guarded against.

Progress in this matter in the United States has been prevented by the past unsuccessful operation of fine screens of American design, which have lacked features essential to success both in design and construction. Designs must be made with due consideration of the results desired. The removal of solids below the flow of the sewage, and the action or travel of the screen against the flow, produce comminution, and are due to incorrect principles. The important features and advantages of properly designed and constructed fine screens are: Low initial cost; low operating and maintenance costs; application to large or small installations; serviceability as fine or coarse screens; maximum effective screen area; operation under various flow levels; minimum loss of head; effective screen seal; slow speed; small space; availability for existing outfall sewers; noiseless and cleanly operation; screen chamber affording no recesses or corners for accumulation of solids; continuous, automatic, and instantaneous removal of solids, avoiding putrefaction; removal of heavy, true suspended, light, and floating matters; recovery of fats and grease; low velocity of travel of screenings through sewage; minimum disintegration of screenings; non-septic tendency in course of treatment; screenings of a consistency easily handled; and possibility of recovering grease and manurial values in screenings.

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The term "screenings" should be accepted as denoting solids removed by fine screens, these being fresh solids containing manurial values. The term "sludge" should denote solids discharged from digestion tanks, with their values spent.

It is acknowledged that screenings possess considerably greater values than sludge, and are delivered in a spadeable form, with a water content of from 75 to 80%, as compared with sludge, in a liquid form, with a water content of 90% or more, necessitating a sludge bed or mechanical de-watering.

The volume of screenings *versus* the volume of sludge may be realized by reference to Fig. 33, which compares their relative volumes, and is based on the data in Table 25.

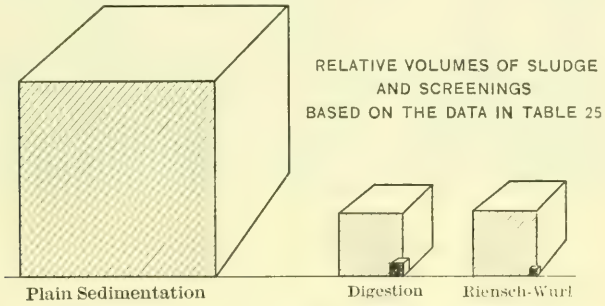

FIG. 33.

TABLE 25.—DATA RELATIVE TO REMOVAL OF SOLIDS FROM SEWAGE BY SEDIMENTATION, TANKS, AND SCREENS.

Method.	Percentage of total solids removed.	Percentage of moisture.	From 1 000 000 Gallons of Sewage.		
			Cubic feet of dry solids.	Cubic feet of wet sludge.	Tons of wet sludge.
Plain sedimentation (2 hours)..	53	95	18.5	370 @ 63 lb.	11.6
Digestion tank (1 hour).....	50	90	17.5	175 " 64 "	5.6
Digestion tank.....	50% digestion.	90	8.8	88 " 64 "	2.8
Digestion tank.....	Dry on sludge bed.	60	8.8	22 " 68 "	0.75
Riensch-Wurl screen.....	50	80	17.5	87½ " 66 "	2.9
Riensch-Wurl screen.....	Burn in destructor (ash = 10% of weight of wet screenings).				0.29

The daily production of screenings, and the resultant necessary disposition thereof, will have on the whole a great effect on the care and operation of treatment plants. A plant of any type will not operate successfully without intelligent and continuous care. Contrary ideas are responsible for the deplorable condition to-day of numerous sewage-treatment plants throughout America. The Engi-

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neering Profession should impress particularly the need of intelligent care and operation.

In Russia, at certain treatment plants, legal requirements prohibit the discharge of solids greater than a certain diameter into public waters; the sewage is screened; the screenings are ground or disintegrated so that they will be of less than the specified size, and are redeposited in the discharged effluent. Such a method is utterly useless, and is on a par with some attempts to discharge undigested or digested sludge with the effluent from treatment plants. Under no conditions should such disposition be considered or permitted. Where such a practice is permitted, a resultant deplorable condition will soon manifest itself.

The method of removing solids from the screen is important; any hoeing or raking action will force them through the meshes or openings. Hydraulic methods are not effective, as they do not remove fibrous matter, and are objectionable on account of splashing and also the additional water content of the screenings. A continuous revolving brush, covered with a hood connected with air suction, is undoubtedly the most effective and sanitary.

In the design and mechanical construction of screens, due consideration has not, until recently, been given to the stresses and strains resulting from service under the hydraulic head frequently encountered under working conditions, with the result that invariably, with American installations, there have been serious difficulties, either leading to abandonment or necessitating reconstruction; these defects the Germans have remedied with screens of several types, so that the service is continuous and satisfactory.

The Riensch-Wurl screen, now in operation at Dresden, Bremen, Eberswalde, and numerous other places in Germany, as well as in Norway, Russia, and France, has, in the past five years, met and fulfilled all the requirements. Its recognition and adoption in the United States have led to the purchase of the invention and its manufacture in America.

In connection with the adaptation of the design of the screen for American service, important features suggested by experienced civil engineers have been incorporated, and have insured its satisfactory operation. It has the largest available screening area, permitting a reduced velocity in the flow; it operates in the flow of the sewage, gently, gradually, and continuously, raising the screened and settled solids to a position where they may drain and be swept from the screen plate by a correctly designed brush, in a manner which absolutely removes the solids without forcing them through the screen openings, as shown by Fig. 34. The design of the brush and its operation may be better understood by reference to Fig. 35.

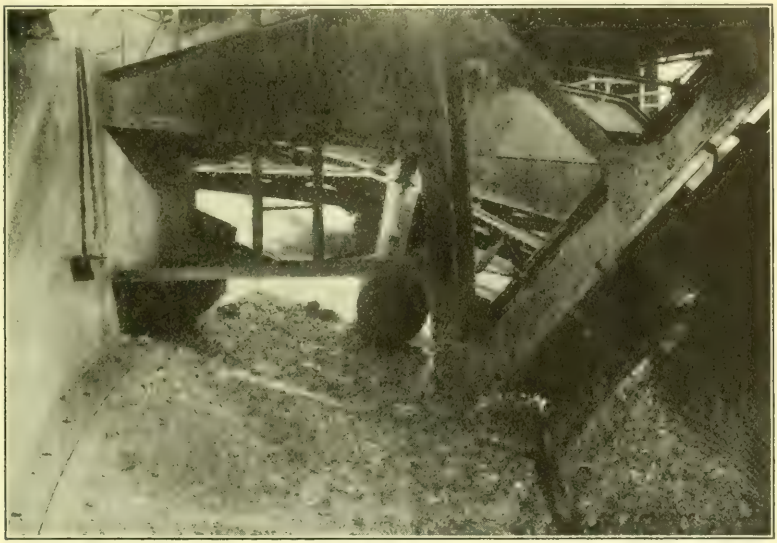

FIG. 34.—SECTION OF RIENSCH-WURL SCREEN, SHOWING SCREENINGS COMING UNDER THE ACTION OF THE BRUSHES.

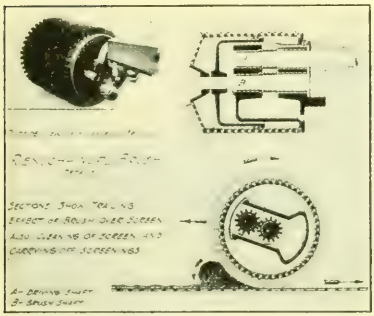

FIG. 35.—SECTIONS OF BRUSH, SHOWING ITS TRAILING EFFECT OVER SCREEN, ETC.

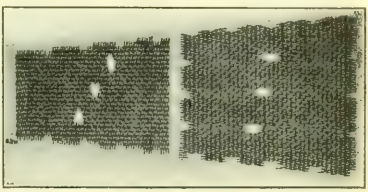

FIG. 36.—SCREENING PLATES OF RIENSCH-WURL SCREEN.

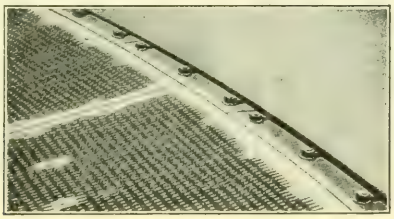

FIG. 37.—ADJUSTABLE SEAL OF RIENSCH-WURL SCREEN.

FIG. 38.—EXTERIOR OF RIENSCH-WURL SCREEN-HOUSE, DRESDEN, GERMANY.

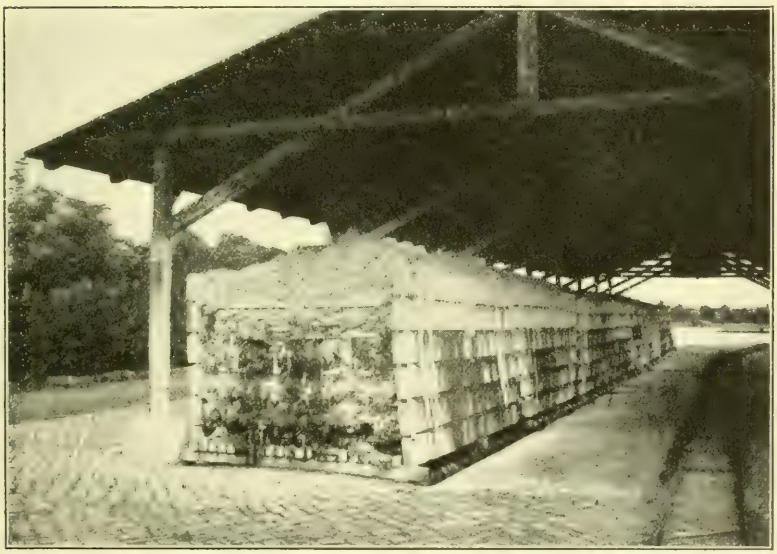

FIG. 39.—SALE SHED AND BINS FOR STORAGE OF SCREENINGS.

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The screen plate is smooth, with tapered openings, insuring free passage without obstruction, as shown by Fig. 36.

The screen acts as a skimmer, in removing a maximum percentage of grease, and, with sewages containing high percentages of this substance, will occasionally require freeing by steam, gasoline, naphtha, or other solvent. The bearings are above the flow, and are easily accessible for inspection and repair. All parts of the screen structure are readily brought above the sewage flow for care and attention. In the bascule design, the entire screen may be raised out of the flow of the sewage.

The power consumption is low, and also the operating speed of the screen, and may be varied and controlled automatically. The brush-cleaning mechanism works in a fixed ratio to the screen speed. The loss of head is a minimum. The screen seal, as shown by Fig. 37, prevents leakage and insures fine screening of the entire flow. At Dresden, Germany, one man takes care of the entire station of four 26 ft. 3-in. screens. The size of the screen-house may be noted from Fig. 38. In design and construction, due consideration has been given to the varied and at times extreme thrust and strains caused by heavy loss of head.

Mr. Allen's compilation in Table 12 was based on data available from tests made under such varied conditions as to render the comparison of no real value. An efficiency of 90% for the band screen at Göttingen is questioned. An efficiency of 63% for the Jennings' screen is the result of operation on stock-yard wastes, and therefore is not comparable with domestic sewage. An efficiency of 10% for the wing screen at Frankfort is attributable to the presence before the screen of an effective grit chamber and to the large openings in the screen, which is of the bar type. At Strassburg and Gleiwitz efficiencies of from 10 to 63% are explainable by the different types of installations. The 53.5% efficiency of the screen at Mainz is the result of experimental operation only. The efficiencies of the Weand screens at Reading and Brockton (42 to 71.3%) are due to the difference in design of the plants.

The efficiency of the Riensch-Wurl screens at Dresden, scheduled at 33.6%, is the result of one method of test. Numerous trials of these screens, in some fifty plants, show from 32 to 86%, averaging more than 50 per cent.

So many factors have such a serious effect on test readings as to render them of no practical value.

Tables 9 and 10 will be better understood with the following explanation: Table 9 shows a cleaning effect of 66.4%, as the average for a month, and Table 10 an average of 33.64 per cent. In comparing the two logs, attention is called to the fact that the screen resolves itself into a classifying apparatus. It will be noticed that

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the solids in the effluent in both cases were about the same, and that the influent varied about 100% in its content of solids. Furthermore, as the author states, the data in these two tables were recorded under differently prescribed methods of sampling, which has a great influence on the results obtained, by screens or any other method of sewage treatment.

The earlier practice at Dresden was to draw samples of sewage, both influent and effluent, in a pail, collect a quantity, and then draw a composite sample, and, with that make a cone glass reading, as is usually made for tanks. For a period of 3 months daily recorded tests varied from 33 to 86%, averaging 66+ per cent.

With the distinction between sludge and screenings, to which reference has already been made, their handling and disposal will be of interest. Tank sludge, with its 90% (or more) of moisture, requires re-handling in a de-watering stage, and is ultimately of no value, so that its disposal is in every sense an item of expense. Screenings containing 80% (or less) of moisture must be handled promptly. Disposal by incineration, as suggested by the author, is sanitary and effective, but the possibility of at least some return is being considered more and more. By-product processes are developing. Grease, ranging from 8% to greater quantities, may be recovered profitably and produce a further de-greased dry sludge by-product, valuable as a fertilizer. Mechanical de-watering of sludge, incineration, and by-product recovery from screenings are all established methods. Fig. 39 shows the screenings at Dresden stored and awaiting delivery to farmers. The city formerly disposed of all its raw screenings for $500 per year; with its by-product recovery plant, the returns have been increased to $2 000 per year.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT.

Discussion.*

By M. M. O'Shaughnessy, M. Am. Soc. C. E.

Mr. O'Shaughnessy.

M. M. O'Shaughnessy,† M. Am. Soc. C. E. (by letter).—The writer has read this paper carefully and confesses himself embarrassed to analyze its motive and value. There has been much discussion in recent years as to the various fields in which the engineer may become active, outside of the exercise of his profession as such, and Mr. Cory's paper certainly invades many foreign subjects into which it has never been even suggested that the engineer should inject himself.

Mr. Cory enters into a discussion of political, sociological, moral, economic, and financial features of a great and varied State, and all the sub-divisions thereof, and it is to be feared that he is so enthusiastic in those realms that he neglects the engineering side of the problem.

One of the earliest precepts instilled into the minds of young engineers, both before and after their entrance into the practice of their Profession, is the necessity of obtaining as exact information as possible on a given subject before proceeding with any independent work thereon. It is evident, however, that this precept has been, to some extent at least, disregarded in the present case, as some of the author's statements are only partly true, and others, so far as the knowledge of the writer goes, are not true at all.

The paper consists chiefly of a résumé of matters which were thoroughly and scientifically discussed and settled at the time the

* Continued from December, 1914, *Proceedings*.

† San Francisco, Cal.

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grant of the Hetch Hetchy reservoir sites, rights of way, etc., to the City of San Francisco was under investigation by Federal Government and other experts.

Mr. Cory attempts to revive a number of dead issues, and questions the engineering and financial policies of San Francisco as if these matters had not already been thoroughly considered and disposed of by a number of distinguished engineers, whose names are recognized throughout the United States—and throughout the civilized world—as authorities in their respective fields of work.

The Hetch Hetchy project has been the subject of a vast amount of painstaking thought and labor on the part of the officials of San Francisco and the experts whose advice was sought by them. The people of the City understand the situation of the municipality with regard to the water supply, and are as thoroughly in accord on the Hetch Hetchy scheme at the present time as they showed themselves to be some years ago, when propositions to bond the City for the acquisition of lands and for the construction of the works were carried by a vote of six to one.

The City has offered the neighboring East Bay communities—Oakland, Berkeley, Alameda, and the smaller cities—every opportunity to join with her in obtaining a mountain water supply. These cities naturally form a part of a Metropolitan District the interests of which will be best served in co-operating with the project of San Francisco. Up to the present time all the Bay cities have been furnished with water by private corporations. For years, the Spring Valley Water Company of San Francisco bitterly fought the acquirement by the City of rights in the Hetch Hetchy region, and by a skilfully planned propaganda created very strong opposition to the City among the irrigationists whose lands are contiguous to the Tuolumne River. This corporation opposition did not cease until the Raker Bill was passed.

The City officials have since agreed with the Company on a purchase price for its property, and this is to be submitted to the voters for ratification at a special election in March, 1915. There is some opposition to the proposal, based on the idea that instead of buying the Spring Valley properties at a price which those opposing the plan consider excessive, the City should proceed at once to the construction of a Hetch Hetchy system, and rely entirely on this source, independent of the existing works of the Spring Valley. The opposition from the irrigation districts to the Hetch Hetchy scheme was modified on the inclusion of conditions, in the Raker Bill, which guarantee to the irrigationists their developed rights and a much larger quantity of water than they were technically entitled to under the State laws for 60 days of the flood period of each year; thus, there is no longer any serious opposition to the Hetch Hetchy project from

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any faction in San Francisco or in the San Joaquin Valley, except from some irreconcilable agitators.

The people of the transbay cities generally favor uniting with San Francisco to form a water district, and such opposition as now exists in those communities has been stirred up by the Peoples Water Company, which is in financial turmoil from imprudent high financing. This Company made futile opposition to the Raker Bill. Its only hope for survival is to keep its customers out of the proposed Metropolitan Water District.

Such is the general situation of the San Francisco Bay region with regard to the mountain water supply. The question of necessity has been settled; the Hetch Hetchy has been found to be the most available source of supply by a board of distinguished and disinterested army engineers, either for San Francisco alone, or for the proposed water district. San Francisco, without waiting for the formation of the Metropolitan Water District, has already begun construction work in the Hetch Hetchy region.

The writer cannot see in what respects the matters touched on by this paper are of sufficient interest to the engineering world to justify its publication by the Society; but, as it has been published, he will, without attempting to reply to it in detail, indicate a few points on which he takes issue with the author.

First, as to competition between the San Francisco Bay cities and Los Angeles: The Bay cities do not regard themselves as competitors of Los Angeles. This statement must not be taken as a confirmation of Mr. Cory's idea that San Francisco is self-sufficient. It means that San Francisco recognizes her southern neighbor's place in the general scheme of things; that the interests of the two cities are dissimilar; that the prosperity of one is helpful to the prosperity of the other; and that San Francisco realizes that increased prosperity in Southern California benefits the State as a whole.

However this may be, competition or non-competition between the Bay cities and Los Angeles could not be a factor in determining the choice of a water supply. Conditions are so different in the two sections of the State that a matter of such small import as the annual water bill could cause little difference to any one who is making his choice of a permanent home, or be a final factor in determining the moral status or commercial triumph of either city. The comparison of sociological conditions in the two communities is no part of an engineering problem. However, personal knowledge and civic pride impels the writer to correct a few statements in Mr. Cory's comparison, which are either wholly or partly incorrect.

In a comparison of growth, on page 2108,* the growth of the San Francisco Metropolitan District from 1900 to 1910 is given as 41

* *Proceedings*, Am. Soc. C. E., for September, 1914.

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per cent. Referring to the Abstract of the 1910 Census, page 62, it is found that the figure should be 45.2 per cent. A comparison of the population curves, Fig. 2, with the figures of the Census on page 62 of the Abstract, shows that in Fig. 2 the population of "Greater San Francisco-Oakland" in 1900 is 25 000 less than the Census figure, and in 1910, 60 000 less; the population of "Greater Los Angeles" in 1910 as plotted is 30 000 less than the Census figure. These discrepancies are not very large, but they work against the northern cities. Furthermore, any comparison between the growth of San Francisco and that of Los Angeles since 1900 must take into account the retardation of San Francisco's growth by the disaster of 1906 and the great apparent acceleration of that of Los Angeles due to the annexation of a number of small cities and towns in the outlying districts.

On page 2117,* it is stated that the San Francisco port charges and regulations are onerous. Referring to a paper by H. M. Chittenden, M. Am. Soc. C. E., entitled "Ports of the Pacific",† the writer would call attention to Table 6, "Comparative Costs to Ship and Cargo in Pacific Coast Ports", and quote from the note at the end of that table:

"This table is the result of a comparative study of the rates in the different ports with a view of determining the relative tax on commerce in each. In order to arrive at a condensed approximation of the cost of terminal charges and a comparison of the principal Pacific Coast ports, a purely hypothetic case has been assumed of a 6 000 net ton register ship, 30-ft. draft, discharging and loading cargoes of 12 000 tons of miscellaneous freight, the ship remaining in port 12 days."

TABLE 7.

	Total cost.	Cost paid by ship.	Cost paid by cargo.
San Francisco	$18 042.50	$1 242	$16 800.50
Los Angeles	21 599.00	407	21 192.00
San Diego	22 475.00	875	21 600.00
Portland	22 280.00	680	21 600.00
Puget Sound	21 800.00	200	21 600.00
Vancouver, B. C.	21 830.00	230	21 600.00

The total cost to ship and cargo in the various ports is shown in Table 7, from which it will be seen that the cost paid by the ship is three times as great at San Francisco as at Los Angeles; the cost paid by the cargo at San Francisco is 79% of that at Los Angeles; and the cost paid by the ship is so small a part of the total cost that the total cost at San Francisco is less than 84% of that at Los Angeles. Furthermore, Table 7 shows the total cost at San Francisco to be

* *Proceedings*, Am. Soc. C. E., for September, 1914.

† *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 208.

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lower than at any of the other five ports mentioned. It is immaterial to both the shipper and the ultimate consumer how the port charges are divided, for that which interests them is the total amount they have to pay.

The term, "Paris of America", applied by the author to San Francisco, is an echo from a political campaign of a few years ago. It was first used in reference to some proposed system of municipal improvements which should make San Francisco the most beautiful city in America and a pleasure resort for the whole continent, but was later turned to the derogatory sense in which it appears in the paper. It died with the campaign, and it is not fair to use such casual expressions in a comparison of the solid merits and achievements of cities.

As to the 2 200 saloons in San Francisco, the Police Commission of San Francisco gives the number of retail liquor licenses now in force as 2 051, which includes some hundreds of restaurants where liquor is sold with meals.

The civic spirit of San Francisco is manifested in the attitude of the people toward public improvements. When shown the necessity or advisability of such works, they have been most liberal in voting bonds, although showing great intelligence and discrimination in demanding proof of the advantage to come from proposed expenditures. A great amount of money has been spent during the past ten years, and more is now being spent, on improvements the cost of which is approximately as follows:

New sewer system, nearly complete	$6 150 000
High-pressure fire system, acknowledged by all experts to be the best in the world, containing 72 miles of mains, covering all the principal business and residence districts, and reducing the insurance rates annually $1 250 000	5 750 000
Municipal street railways, 25 miles of double track, with rolling stock and car barns	5 500 000
Asphalt plant	60 000
Civic center	8 800 000
Schools	7 500 000
Hospitals	3 000 000
Fire and police stations	510 000
Public library lands	740 000
Hall of Justice and County Jail	1 350 000
Hetch Hetchy water supply, lands and preliminary work	2 000 000
Docks	9 000 000
Streets and pavements	15 000 000
New parks and public playgrounds	1 360 000
Traffic tunnels	4 600 000
Total	$71 320 000

Mr. O'Shaughnessy.

In addition to these municipal enterprises, about $11 000 000 has been expended on the Panama-Pacific International Exposition, the total expenditures on which will amount to about $18 000 000. Of this amount $7 500 000 was subscribed by corporations and individual citizens of San Francisco, and $5 000 000 was raised by the issue of municipal bonds.

The total amount of municipal bonds sold during the past ten years is $47 701 200. The City's bonds have generally commanded premiums, so that the total amount of money realized was about $49 253 000. The total amount authorized by the voters, but not yet sold, is $45 832 800, which includes $43 428 000 for the Hetch Hetchy water supply. The harbor bonds are a State issue, to which the citizens of the State contribute nothing, as the interest and sinking-fund payments come from the port revenues. The difference between the total cost of improvements mentioned and the total amount of bonds sold is accounted for by premiums on bonds and by payments for work out of current revenues and by assessment districts.

The municipal street railways constitute the first essay of San Francisco in the municipal ownership of public utilities, and the results are very gratifying. The first line was opened to traffic on December 28th, 1912, and at present there are 18 miles of double-track railways in operation, 4.5 miles under construction, and 2.5 miles being planned. The gross receipts during August, 1914, averaged $3 186 per day, and the operating expenses, $1 597 per day. The amount invested in the roads now in operation is about $4 000 000, on which the interest charge is about $525 per day. Liability insurance costs about $45 per day. Depreciation is estimated at 18% of the gross receipts, or $573 per day. The total daily expenses of the roads thus amount to $2 740, leaving a balance of $446 per day, which is a rate of more than $160 000 per year, or 4% of the investment—much more than the amount of the taxes that a private corporation would pay—available to be applied to extensions of the lines. This augurs well for the City's ability to manage successfully its own water system and the accompanying power development. Incidentally, it may be mentioned that San Francisco is the first city in America to achieve financial success in the operation of a municipally owned street railway system.

The writer apologizes for having taken up so much space with matter entirely irrelevant to the subject in hand, his excuse being that he wishes to correct any misapprehensions which might become fixed in the minds of the readers of the paper, if the statements contained therein were permitted to go uncorrected.

As to the water supply situation in the Bay cities, Mr. Cory presents considerable discussion on the present and possible future sources

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for San Francisco, beginning with the Spring Valley Water Company's system. For years there has been much investigation as to both the ultimate and the available yield of the Spring Valley sources. Many reports and estimates have been filed by various parties—some representing San Francisco or the East Bay cities, or both; others working in the interests of one or more of the water companies supplying those cities. In matters of this sort it usually happens that investigators of the latter class are very optimistic in their estimates, and those of the former class are more conservative. Table 1 shows the estimated yield of the various Spring Valley sources. The grand total yield of the Water Company's Peninsular and Alameda systems is estimated by Messrs. Herrmann, Schussler, Chittenden, and Powell, at about 220 000 000 gal. per day. These engineers were all engaged on the Spring Valley side of the controversy before the Secretary of the Interior regarding the City's proposed Hetch Hetchy development. Their estimates were reduced 50% by the U. S. Army Engineers, and, furthermore, it was shown that the yield was not obtainable, except at prohibitive cost, for municipal purposes. Mr. Dockweiler, representing the City in the same case, in his American-Cosumnes report, gave an estimate, also shown in Table 1, of 109 500 000 gal. per day—about half the amount of the estimates of the engineers on the other side. Later investigations prove even this to be excessive.

The run-off and discharge records mentioned by Mr. Cory on page 2132* are not by any means all that could be expected of run-off and discharge records. Those on Calaveras Creek consist mainly of a series of records of depth and width and crude float observations over a very short course, taken by a watchman. No serious attempt at scientific stream gauging has been made until very recently. Members of the staff of the City Engineer of San Francisco, with the co-operation of the Spring Valley Water Company, are now making such measurements of stream flow from the Company's sources as will make possible more precise estimates of the possible water development from those sources than can be made at present from the rough and ready measurements of the past.

The same degree of uncertainty which exists with regard to the Spring Valley sources, and the same wide variation of estimates of possible yield, are encountered in the study of the sources from which the transbay cities draw their water supply. The situation in Alameda County is complicated further by the fact that the various streams, the water of which it is proposed to take for domestic supply, are the same streams that replenish the underground water supply of the valleys below, and any further withdrawal of water, beyond the quantities now taken, will be bitterly opposed by the farmers who use the underground supply for irrigation.

* *Proceedings*, Am. Soc. C. E., for September, 1914.

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Yet, Mr. Cory, unmindful of the differences between engineers who have made long and careful study of the various sources, and not heeding the possible legal complications, states positively on page 2136* that the total ultimate safe draft for municipal purposes for San Francisco and the Peninsula, the East Bay region, and Marin County and the North Bay region is 231 000 000 to 256 000 000 gal. per day, allowing a variation from minimum to maximum of only 25 000 000 gal.

It is certainly absurd for any one to assume that the engineers for the Bay cities have not given thorough investigation and consideration to the possibilities of the local sources of supply. In doing so they have analyzed all phases of the question, and have given due weight to inaccuracy of records and possible legal complications, and also to the fact that the development of some portions of the local sources would be so costly as to be prohibitive, unless there was absolutely no other available source.

For example, the coast streams of the Spring Valley Water Company were credited by Mr. Dockweiler, in his American-Cosumnes report, with a yield of 31 700 000 gal. per day, but to develop this water would be so expensive that ordinary financial prudence would demand that the money be used toward the development of the large supply, at less cost, which it is ultimately proposed to bring from the Sierra Nevada.

It is clear, to those who have investigated the situation from a fair and impartial standpoint, that the supply which can be developed economically from the local sources will be scarcely more than sufficient to keep pace with the normal growth of the cities and the suburban communities tributary thereto until the mountain water can be introduced.

Whether the problem is one for San Francisco or for "Greater San Francisco-Oakland", the answer is that San Francisco and Oakland are both "in the same boat" as regards water supply; both need immediate extensions, both are suffering from the exploitation of private speculative corporations, and San Francisco would welcome the cooperation of Oakland in the development of her water supply; but, if Oakland does not decide to assist, San Francisco must, of necessity, go ahead by herself.

Under the discussion of whether "Greater San Francisco-Oakland" can afford a Sierra supply, Mr. Cory, on page 2139,* asks: "Can San Francisco afford the luxury of a Sierra water supply at an immediate expense of more than 10% * * * of its present assessed valuation?" (The present assessed valuation is $646 700 065.) Also, on page 2178,* he states: "The City has revised the estimated cost of the first

* *Proceedings*, Am. Soc. C. E., for September, 1914.

160 000 000 gal. per day installation from Mr. Freeman's figure of $37 501 400 to $64 000 000, and probably the figures on the other installations as well." Mr. O'Shaughnessy.

The writer is at a loss to understand these statements. Mr. Freeman's estimate of the total cost of the initial installation, not including the cost of a city distributing system and the permanent power development, is given on page 160o of his published report of July, 1912:*

> "Total cost of supply, varying from maximum of 240 million gallons daily to minimum of 160 million gallons daily in extreme drought, delivered to Irvington gate house overlooking San Francisco Bay, with 50 millions delivered into San Francisco 3 miles from City Hall, and 50 millions into Crystal Springs Reservoir when desired.... $36 981 000
> "And also with 60 million gallons daily ready for Oakland, San Jose, etc."

Mr. Freeman's estimate of $36 981 000 (not $37 501 400), on the basis given by him, still stands. Thus far, no radical departure from the Freeman plans is proposed, except that it may be considered advisable to install initially a smaller pipe line, 45 miles long, across the San Joaquin Valley for a supply of 60 000 000 gal. per day, and to develop at once 50 000 hydro-electric horse-power. The cost developed by Mr. Freeman was based on the assumption that common labor will be paid a rate of $2.25 per day of 8 hours. The Charter of San Francisco requires that common labor be paid $3 per day on municipal work. It is still an open question whether this rate will be held to apply to all work outside of the City as well as that within the City limits. If it should be so, the effect will be to increase the cost of the whole project perhaps 20%, bringing the cost of the system, if built according to the Freeman plans, to about $45 000 000. (These figures do not include a city distributing system.)

On page 2178† the statement occurs: "There are about 100 miles of steel pipe, much of it under heads as high as 800 and 900 ft." This is erroneous, as the Hetch Hetchy aqueduct, in the Freeman plans, from the Early Intake to San Francisco, contains 66 miles of tunnel and 87 miles of pipe. The highest head (except in the penstocks of the power-houses) is near Oakdale, at the east end of the San Joaquin Valley crossing, and is considerably less than 600 ft. Mr. Cory is 50% in error here.

In calculating on other sources of water supply, the same prices of labor and material were used as in the estimate of cost of the Hetch Hetchy scheme, with the exception of the estimate for the filtered

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* "The Hetch Hetchy Water Supply for San Francisco, 1912: Report."

† *Proceedings*, Am. Soc. C. E., for September, 1914.

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supply from the Sacramento River, in which Mr. Hazen used $3 per day of 8 hours as the wages of common labor. Thus, the question of unit prices does not enter into the comparison of various sources.

This matter was thoroughly understood by the Board of U. S. Army Engineers which acted in an advisory capacity to the Secretary of the Interior regarding the grant of the reservoir sites and aqueduct rights of way. The members of this Board were selected as competent and impartial judges, and not only compared the estimates submitted to them, but made their own independent estimates on the same basis. These covered nearly all the possible sources mentioned by Mr. Cory on pages 2162 to 2176,* including Coast Range sources, supplies from the delta of the Sacramento and San Joaquin Rivers, and Sierra sources from the McCloud River to the Tuolumne. They heard or read all the testimony submitted by the City of San Francisco, the Spring Valley Water Company, and promoters interested in various possible competing sources of supply, and confirmed the judgment of the City's engineers that, considering the problem from the standpoints of both engineering and economics, the Hetch Hetchy is the most desirable. To quote their own words, from page 50:†

"The project proposed by the City of San Francisco, known as the Hetch Hetchy project, is about $20 000 000 cheaper than any other feasible project for furnishing an adequate supply."

Relative to this matter, the following extract, from page 14 of that report, is of interest:

"In one important respect the situation in California requires special consideration. * * * In California all water has great value. Due to the large extent of arid and semiarid land that can be made fertile by the use of water, irrigation is assuming great importance; due to lack of coal and the opportunity for economical water-power development the use for the latter purpose will surely be greatly extended. In a relatively few years practically all available water will doubtless be appropriated for one or the other purpose, and it will then be possible to obtain it for municipal use only at great cost and damage to existing communities and industries. It is, therefore, necessary to-day for the cities of California to look further ahead than in most other parts of the country, and to take such steps that in the future when they may need the water they shall have the right to it. For this reason it is believed that in making provision for the future supply of San Francisco and other bay cities a source should be selected if possible that is capable of supplying the needs of the communities for the balance of this century. Such a course would seem both wise and reasonable, provided it involves no sacrifice of economy."

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* *Proceedings*, Am. Soc. C. E., for September, 1914.

† "Hetch Hetchy Valley: Report of Advisory Board of Army Engineers to the Secretary of the Interior," February 19th, 1913.

Mr. O'Shaughnessy.

This answers the question as to whether or not the Bay cities can afford a Sierra supply. Their material, social, and commercial development depends on their obtaining additional water; the Hetch Hetchy has been shown to be the most economical and adequate source; and between the alternatives of providing additional pure water supply by the community or civic stagnation under existing conditions, there can be but one answer to the question.

The Raker Bill, which grants to San Francisco the necessary reservoir sites and aqueduct rights of way through the public lands, was a compromise between the conditions on which the City first sought the necessary rights for the development of the Hetch Hetchy water supply, and those which the irrigationists sought to impose. In framing the bill due regard was also given to the consideration that the National Park should not be treated in any way that would detract from its status as a pleasure ground for the public. A great deal of thought was devoted to the formulation of this bill by the City's engineers and attorneys, who did not wish at any time to take from the irrigationists any water that they might usefully apply to their lands and be legally entitled to. The City has always realized that its prosperity and that of its back country are to a large extent mutually dependent, and, further, does not wish to antagonize the irrigation districts by taking from them something which they feel belongs to them. The demands of the irrigationists have been increased several times since the City made its initial application to the Secretary of the Interior, and the final result was that the City consented to the practical guaranty that the irrigationists should have water to the extent now legally diverted by them in their two canals, Turlock and Modesto, namely, the full natural flow of the Tuolumne and its tributaries up to 2 350 sec-ft. at La Grange diverting dam when the districts can beneficially use it, and 4 000 sec-ft. from April 15th to June 14th of each year.

This arrangement, however, was only consented to after the City's engineers had thoroughly investigated the available water supply of the Tuolumne, revised their former figures, and found that, even with the restrictions imposed, there would still be sufficient water to fulfill the conditions and leave the City's own future supply up to 400 000 000 gal. daily for domestic purposes unimpaired. In confirmation of this, the following finding of the Army Board is conclusive:

"The board is of the opinion that the use of the Hetch Hetchy Valley as a reservoir site is necessary if the full flow of the upper Tuolumne is to be conserved.

"The board further believes that there will be sufficient water if adequately stored and economically used to supply both the reasonable demand of the bay communities and the reasonable needs of the Turlock-Modesto Irrigation District for the remainder of this century."

Mr. O'Shaughnessy.

The Raker Bill, as passed, has been reviewed by Mr. Freeman, who was one of the principal consulting engineers for the City. On March 21st, 1914, he submitted a report to the City Attorney and the City Engineer entitled, "Reservoirs Ultimately Required for a Hetch Hetchy Water Supply of 400 Million Gallons Daily for San Francisco under the Terms of the Raker Bill". The following is quoted from that report:

"I submit herewith my report upon the additional burden imposed upon the City of San Francisco by increased water rights given to the irrigation districts by the terms of the recent grant by Congress to the City of San Francisco, of rights of way and reservoir sites in the Hetch Hetchy region, by House Bill No. 7207, known as 'The Raker Bill'. I am pleased to report that a most painstaking study proves that the City can never suffer seriously from this cause, and the conclusion is justified anew, that it was ten-fold better for the City to make these concessions than to imperil the passage of the Hetch Hetchy bill, and that the total storage of 190 billion gallons, or say in round numbers, 200 billion gallons, found necessary for a dependable water supply of 400 million gallons daily in a series of years of lowest rainfall ever yet known, by my new computations, can be had in the City's three reservoir sites, Hetchy, Eleanor, and Cherry, with the addition of Vernon Lake and Poopenaut.

"Altho the terms of the grant are now fixed beyond change, it has appeared wise to check up the hurried computations made while the Bill was being negotiated before Congressional committees last Summer and review all data in the most painstaking way, particularly since the wisdom of certain conclusions reached by the City's representatives has been questioned by certain good citizens before one of your most important civic organizations, and also because there is now available the daily record of gagings at the City's dam sites at Hetchy and Eleanor thruout the remarkable severe drought of 1912-1913, to which the priorities specified in the Raker Bill can be directly applied, affording a more variable and convincing demonstration."

The additional burden on the City, due to the terms of the bill, is seen from this report to be merely a matter of constructing some comparatively small and inexpensive reservoirs in addition to the Hetch Hetchy, Eleanor, and Cherry Reservoirs, which have been contemplated all along.

There are reservoir sites available to San Francisco and to the irrigation districts, with capacities which total much more than the maximum storage which will ever be required.

The Raker Bill sets aside for the use of the City all reservoir sites above the mouth of Jawbone Creek (a tributary entering the main Tuolumne River about 15 miles below the Hetch Hetchy dam site), leaving the irrigationists the right to develop storage below

that point. The following reservoir sites are thus available for San Francisco: Mr. O'Shaughnessy.

	Capacity, in acre-feet.
Hetch Hetchy Valley (300-ft. dam)	344 000
Lake Eleanor (250-ft. dam)	288 800
(Mr. Freeman figures on a 150-ft. dam giving 37 000 000 gallons or 120 000 acre-ft. capacity.)	
Cherry Valley (150-ft. dam)	56 800
Vernon Lake	47 900
Wilmer Lake	5 000
Tilden Lake	27 800
Emigrant Lake	14 300
Huckleberry Lake	52 200
Poopenaut Valley	44 300
Cherry Mouth (at junction of Tuolumne and Cherry Rivers)	8 200
Total	889 300

or 273 000 000 gal., which is 73 000 000 gal. more than is necessary, in Mr. Freeman's opinion, as previously quoted, for the City's development of 400 000 000 gal. per day.

Below Jawbone Creek, and contiguous to the Tuolumne water-shed, the following sites remain:

	Capacity, in acre-feet.
Empire Mills Reservoir site	35 000
Coffin Hollow	12 000
Lily Lake	15 000
Cottonwood	25 000
Ackerson	26 000
Stone	26 000
Fernbrook	26 000
Heardin	25 000
Big Creek	12 500
Moccasin Creek	12 000
Upper La Grange	200 000
Dry Creek	325 000
Davis Reservoir (already constructed)	48 000
Dickinson Reservoir	60 000
Dallas and Warner Lakes (storage of 30 000 acre-ft. already developed)	60 000
Total	907 500

or 278 000 000 gal.

Mr. O'Shaughnessy.

All these reservoir sites, with the possible exception of a few of the smallest, are capable of development at an economical cost per acre-foot stored.

Pending the construction of additional reservoirs by the irrigation districts, the Raker Bill provides that whenever the districts desire water in excess of that to which they are entitled under the terms of the bill, the City, on the demand of the districts, shall sell such quantities of stored water as may be needed for the beneficial use of the districts—without impairing the supply for domestic requirements—at such a price as will return to the City the actual total cost of providing such stored water, including a fair proportion of the cost to the City of the conduit, lands, dams, and water-supply system included in the Hetch Hetchy and Lake Eleanor sites. It is provided, however, that the City may require the irrigation districts to purchase a minimum quantity of stored water, and shall not be required to furnish more than a maximum quantity, in any calendar year, the minimum and maximum quantities to be fixed by the Secretary of the Interior, keeping in mind the superior needs of domestic use; and, further, the City is not to be required to furnish more than the minimum quantity until the districts shall have first drawn on their own stored water to the fullest practicable extent.

No hardship on the City is to be apprehended from the obligation under these terms to furnish water to the irrigationists. The latter have already developed storage amounting to 78 000 acre-ft., which is capable of expansion, as already shown, to a final total of more than 900 000 acre-ft. Their dams and appurtenances are not and need not be of the relatively expensive type required for the purposes of a domestic water supply, and, therefore, the provision that the water purchased by them shall be paid for at a price which will include a fair allowance for the cost of construction should certainly induce them to construct their own reservoirs.

Furthermore, the irrigation districts have the option to purchase surplus electric power which the City may not need, for the use of any land owner or owners therein for pumping sub-surface water for drainage or irrigation at cost. This will make economically available to the irrigators the vast underground water supply which lies only a few feet beneath the ground surface but is as yet practically untouched, owing to the prohibitive cost of steam power in small units and electric power purchased from corporations whose prices include the profits of their stockholders.

The districts have already spent considerable money in efforts at drainage of sub-surface water due to the rising of the ground-water from over-irrigation, and a generous supply of power to aid them in this policy will far outweigh any sentimental objections to San Francisco's storage of waste flood-waters on the higher mountain levels.

Mr. O'Shaughnessy.

It is not within the range of probability that if during any year the City's supply of stored water should be brought so low as to make a water famine imminent, she should still be compelled to furnish water to the districts, particularly if they had not used the same diligence in developing the reservoir sites available to them that would be expected of the City both under the terms of the Raker Bill and under the dictates of ordinary foresight and prudence.

Turning now to the conclusions on pages 2185 to 2187,* the writer will endeavor to answer, from the standpoint of a citizen of San Francisco and, to the best of his ability, from the standpoint of a citizen of the proposed Metropolitan District as a whole, the questions that Mr. Cory propounds to the citizens of San Francisco.

The cities of the Metropolitan District should naturally develop together, and all the cities concerned are facing a shortage of water; their development depends on their securing an additional supply at once; their interests are interwoven—the problem of supply is certainly one for a metropolitan water district to handle.

As already stated, San Francisco has attempted to get the transbay cities interested with her in securing additional water supply. Due doubtless to the selfish interests at work, those cities have not yet responded to San Francisco's invitation. There is little doubt, however, that they will eventually do so. It happens, however, that the most available source of supply for the Metropolitan District is also the most available source for San Francisco alone, and as the demand for additional water is on us, San Francisco must proceed independently with her development, until such time as the transbay cities may see fit to enter into the work and share the burden with her.

The studies which have been carried on during the past few years have convinced the City's engineers that the economically available increase of the water supply from the Spring Valley sources is hardly more than sufficient to cover the present unsatisfied demand and the natural increase of consumption during the period of construction of the Hetch Hetchy works.

The Hetch Hetchy scheme has been the subject of very serious consideration by many engineers who, after comparing it with all other available sources of water supply, have endorsed it. The engineers who have questioned it, with the sole exception of those employed by the irrigation districts, have been the representatives of special competing interests, hoping to dispose of their own private holdings for their own private gains. The Raker Bill, as passed, imposes on the City a slight additional burden to that which it has always expected to sustain, in the form of obligations not to interfere with the developed water supply of the irrigation districts, and the City, by consenting to the conditions of the bill, has eliminated the unjust accusation

* *Proceedings*, Am. Soc. C. E., for September, 1914.

Mr. O'Shaughnessy.

that she was trying to gain something at the expense of the irrigationists.

The securing of the Hetch Hetchy water supply is not a matter of sentiment with the people of San Francisco in the limited sense in which the word "sentiment" is used by Mr. Cory. The sentiment of the people is for the development of their City. Their accredited representatives have shown, to their satisfaction, that the Hetch Hetchy is the best source of water supply available, and they are willing to invest in it, not "sums so vast that many other requirements more vital to the interest of the City must be sacrificed or skimped", but the reasonable and necessary cost to secure the water supply which is essential to the growth of the City and the expansion of its enterprises.

The acquisition of the present water systems of San Francisco and of the Bay cities, either by a Metropolitan District, or pending the formation of such a district, by the cities acting independently, is a very desirable step toward the development of a well-balanced metropolitan system.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA.

Discussion.*

By Gardner S. Williams, M. Am. Soc. C. E.

Mr. Williams.

Gardner S. Williams,† M. Am. Soc. C. E.—The illustration of the draft-tube showing the cradle (Fig. 39), takes the speaker back to the construction which it fell to his lot to design and supervise at Sault Ste. Marie, in 1906, for the Edison Sault Electric Company. There the so-called cradle was used for setting the draft-tube, and one of the first curved concrete draft-tubes in the United States was built. A method utilized there—by the same contractors who constructed the plant described by the authors—was that, instead of wooden lagging on the form, hard wire cloth of ½-in. mesh was used, which would conform itself readily to warped surfaces, the main ribs being placed in the same manner as indicated by the authors. This hard wire cloth was tacked over the ribs, and outside of it canvas was stretched. This construction proved to be appreciably cheaper than wooden lagging, and secured very nearly as good results. Of course, the concrete surface was not quite as smooth as that against the smoothly planed wooden form would have been, but it was smooth enough for the purposes required.

In that same plant, there was introduced probably the first concrete scroll wheel-pit used in connection with a water-wheel installation of low head, at least in the United States.

Considering draft-tubes brings up the point that there is considerable importance in their form. What may seem like a comparatively insignificant change in form may have an appreciable

* Continued from December, 1914, *Proceedings*.

† Ann Arbor, Mich.

Mr. Williams.

influence on the performance of the wheel. In relation to this, the speaker had an interesting experience within the past 6 months. About a year ago, a contract was made with a prominent wheel maker for a pair of wheels to be mounted in identical pits, and to be of identical capacity and characteristics. The contract called for a forfeit if the performance fell below the maker's guaranty of efficiency, which was 3% below the Holyoke rating of the model wheel, and a bonus if it were above the Holyoke test. When the plans of the draft-tubes were submitted to the maker, he declined to guarantee the performance of his wheels with the draft-tubes as designed, and sent a drawing of what he would accept. The speaker had some rather serious objections to adopting the type submitted, but it was arranged that one of the tubes should be built according to that plan and one according to the original design, and that the performance of the wheel would be determined on the tube built according to the maker's design. Dynamometer tests were made, the water being measured over a weir, and the draft-tube according to the original design gave an efficiency 3% higher than that of the other. The maker suspected that there was something in the pull of the dynamometer that might cause an interference with the action of the wheel, and, when the generators were installed, another test was made which gave similar results. Vacuum gauges were then attached at the wheel outlets, and it was found at once that the tube constructed according to the original design was showing a materially higher vacuum than the other. The final result gave about 7% more power and 5% better efficiency for the tube designed according to the speaker's theory.

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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

FUNDAMENTAL PRINCIPLES OF PUBLIC UTILITY VALUATION.

Discussion.*

By Messrs. J. P. Newell, Charles Rufus Harte, Alexander C. Humphreys, H. F. Dunham, Stuart K. Knox, and C. E. Grunsky.

Mr. Newell.

J. P. Newell,† M. Am. Soc. C. E. (by letter).—In this admirable paper, Mr. Alvord has failed to observe an important distinction. The Courts generally have assumed that the principles which have been established for valuation in condemnation cases are also applicable to rate regulation. In fact, there is a wide difference.

In condemnation, the owner is to be deprived of property producing present or prospective income. The duty of the appraisers is to fix the value of the property in accordance with the income.

Under regulation, the duty of the public service commission is to determine, not what the value of the property is, but what it ought to be. The value of a property devoted to the production of an income can be measured only by the income which it produces; consequently, when the commission decides that the property of a utility is worth a certain sum, and establishes rates which will bring a fair return on that amount, it has thereby established the value of the property. It has not found that the property is worth that sum, it has made it worth it. It will remain at that value as long as the conditions are unchanged. The determination of value for rate regulation, therefore, is not the ascertaining of an existing fact, but the making of a judicial decision. The duty of the appraiser is not to work out a set of mathematical calculations which will give the desired result, but to place before the commission the facts on which its decision is to be based.

* Continued from December, 1914, *Proceedings*.
† Portland, Ore.

Mr. Newell.

A clear recognition of this principle will aid in avoiding some of the errors pointed out by Mr. Alvord. Particularly is this true in the consideration of "going value" or development cost. The commission is under no obligation to capitalize past deficiencies in income, nor to allow an increased rate that will exactly offset them. It is its duty to take into account the causes of such deficits, the skill and judgment shown in the management of the property, the financial conditions prevailing in the community, and the returns earned by equally hazardous private enterprises, and fix such value, or rate of return—which amounts to the same thing—as will satisfy the reasonable expectations of the investor. In this, as in all other matters pertaining to the regulation of rates, no amount of technical or legal ability can take the place of sound business sense and experience.

The writer agrees with Mr. Alvord that, in rate regulation, we must consider original cost, cost of reproduction, depreciation, probable obsolescence, past and present income, market value of stocks and bonds, in short, everything which would be taken into account by a prudent investor in a private property. He is nevertheless of the opinion that the factor which should usually have the greatest weight is the amount wisely invested, modified by subsequent changes in the prices of land, labor, and materials; in other words, the reproduction cost by the historical method.

Mr. Harte.

CHARLES RUFUS HARTE,* M. AM. SOC. C. E.—In this paper Mr. Alvord makes a notable contribution to the growing literature on valuation, and one particularly commendable for its breadth of view and freedom from bias.

That there are some who will take issue with the author is made evident by the paper, entitled "The Valuation of Public Utility Property", by J. H. Gandolfo, Assoc. M. Am. Soc. C. E., but the majority of engineers engaged on or interested in valuation work can find little to which exception may be taken.

Considering these minor points, it seems regrettable that the list of definitions was not further extended; no small part of the confusion on the subject is directly due to the use, by different writers, of the same expression with different meanings.

Of those given, that of "Depreciation" is apparently a slip of the pen. It should be "A loss in value", etc., the "lessened" value, as it appears, being actually the complement of depreciation.

It is true that in certain Court decisions "value" has been defined as "measured in money", but this is not necessarily true, and the first phrase of this definition would be more strictly correct if the last three words were stricken out.

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* New Haven, Conn.

Mr. Harte.

So, too, in the definition of "Amortization", the last four words restrict too closely. Amortization—literally, prevention of death—is applicable to operating death of any sort, whether due to obsolescence, inadequacy, or what not.

As exactness and clearness in definitions are of the utmost importance, the use of the expression "money or its equivalent" in defining "Past Cost", and of "capital or its equivalent" in defining "Investment", intimates a fine distinction between the two, which seems hardly warranted.

In recognizing the "law as the foundation", there is a peculiar condition which should not be overlooked. The Constitution sets forth the principles on which is founded our national existence; the interpretation of the Constitution lies with the Courts; and the adherence of the Courts to precedents in a measure prevents review, unless material error can be clearly established.

There is, however, another and even more effective method of securing what is practically a reversal of a Supreme Court decision. The Constitution fundamentally is the expression of the beliefs and attitude of the Nation; if there are changes in those beliefs, or if a change is necessary in the instrument to secure proper expression of the beliefs or attitude, not only is it possible to effect such change, but seventeen Amendments testify to actual modifications; and not the least interesting feature is that the protection against confiscation of property without due compensation, now enjoyed, was not an expressed element of the Constitution as originally adopted, but is secured through two such Amendments, the Fifth and the Fourteenth; so that particular necessity exists for education as to fundamental principles, not merely as regards engineers engaged on or interested in valuation, but especially as regards the people as a whole. The mechanism by which the Constitution is amended is complicated and slow in operation, to ensure that sudden whim or passion shall not effect mischief, but it is entirely within the bounds of possibility that a change in public sentiment might result in another modification, whereby "property" would be defined so as to withdraw public utilities in part at least from the protection now granted by the Fifth and Fourteenth Amendments.

On page 2422,* in enunciating the Seventh Principle, the author speaks of "computed depreciation". For accounting purposes, or to bring figures found as of different dates to a common time, it is often necessary to compute extensions based on determined facts, but unless the fundamental data are determined in the field for the specific case, and the time interval is reasonably short, "computed depreciation" is, at least, as likely as not to be incurred.

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Harte.

There recently came to the speaker's attention the case of a property which in 1908 was reported, by a well-known engineer, to have roundly a cost to reproduce new of $40 000, with a depreciation of $25 000. A year later $20 000 was put in, in additions, and $5 000 in deferred maintenance, which apparently meant $60 000 reproduction new and $40 000 "less depreciation", yet five years later, with practically no maintenance, the Public Service Commission, after a field investigation, concurred with the speaker in a "cost less depreciation" of $50 000 instead of the $30 000 which would have resulted from following the theoretical—and, as it proved on test, inaccurate—computation of the first valuation. Although this was an extreme case, a very appreciable difference will almost always be found between the depreciation as computed and the actual conditions.

In the illustration of the appraised horse, the unresolved doubt as to the kind obtained sets forth exactly the predicament of the valuer who has used "Life Tables"; he has secured an average value, but, in order to compare it with the actual utility, it must be given the specific details that will establish its position.

On page 2431* there is a statement—that "capital has properly treated the public"—which the author can hardly mean without exceptions. There have been a great many cases where capital has properly treated the public, but the unhappy tendency to restrictive and harassing legislation, which, fortunately for all, apparently is now being succeeded by a more rational attitude on the part of the public, is largely if not entirely due to the "public be damned" policy of some utilities.

The attitude of the utility to the public is an element of its value, however, for a condition of ill will means sooner or later unusual expenditure, either to overcome it or, less wisely, to secure necessary or desirable privileges in spite of it, precisely as good will makes for the contrary.

These comments, however, in the main touch only minor details; as a whole, the paper is admirable.

Mr. Humphreys.

Alexander C. Humphreys,† M. Am. Soc. C. E.—Referring to Mr. Dow's remarks as to whether we can determine the cost of a property by going to the books: Many seem to think that any information which one has a right to ask for can be obtained from the books of account. Nothing is farther from the facts, for the systems of classification vary greatly, even in the same set of books, from year to year; and in the past it was in many cases considered conservative and highly proper to charge the capital account with parts of the operating cost. The result is that many properties are now

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† New York City.

represented on the books of account by amounts which are below their cost. Mr. Humphreys.

The speaker finds much in Mr. Alvord's paper which he would be glad to discuss in detail, but will confine his remarks almost exclusively to that much-discussed and much-misunderstood subject of "depreciation", so-called.

Before taking up this subject, he wishes to enter a protest regarding the position taken by Mr. Alvord. On page 2393* is found the heading, "Law—The Foundation." If it is the foundation, then we must understand what is the law. We naturally turn to the lawyers and the judges for the required information; but we find that they are far from being in agreement as to the law in respect to the valuation of railroads and other public utilities. The speaker agrees with Mr. Alvord that a "knowledge of the interpretation of the law by the Courts in their decisions and opinions" is of great importance to all. He also agrees with him that the Courts "as well as economists and engineers, must accept fundamental law as it stands; their duty is to tell us what this fundamental law means".

Unfortunately, however, the judges and commissioners (for the latter do a large amount of interpreting also) are not infallible. The result is that there is no such positive foundation for us to rest on as Mr. Alvord seems to suggest.

These men are human, like the men of other professions, and frequently they are far from being judicial in their consideration of the questions submitted to them. We must not forget that the judges are lawyers by profession, and that the lawyer, by training, is an advocate.

Our difficulties are greatly complicated because of the opinion which prevails generally throughout the country that our conditions can be improved by the enactment of a multiplicity of additional laws, rather than by the enforcement of the laws now on the statute books.

Senator Elihu Root, certainly a man who can speak on this subject with authority, recently made the following statements in a public address:

"We make too many laws. Our National and State Legislatures passed 62 014 statutes during the five years from 1909 to 1913, inclusive.

"During the same five years 65 379 decisions of the National and State Courts of last resort were reported in 630 volumes. Of these statutes, 2 013 were passed by the National Congress; and of these decisions, 1 061 were rendered by the Supreme Court of the United States.

"Many of these statutes are drawn inartistically, carelessly, ignorantly.

"Their terms are so vague, uncertain, doubtful, that they breed litigation inevitably."

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* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Humphreys.

Under these circumstances, should we meekly submit to Court opinions, particularly when we remember how conflicting those opinions are on any one important question.

We have to submit to the highest Court when it rules, but that by no means implies that we must give up our efforts to present the truth as we see it; and on not a few questions we engineers are better qualified to present the truth than are the lawyers. This has been publicly acknowledged recently by Judge Prouty, formerly Chairman of the Interstate Commerce Commission, and now Chairman of the Board appointed by the Government to supervise the valuation of our railroads.

As the speaker analyzes the present unfortunate conditions in the United States, including placing in the hands of public service commissions authority in the three functions of government, which we, in another breath, declare must be kept separate, the speaker sincerely believes that much of our trouble has come from the almost servile attitude of the representatives of vested interests toward the men who are placed over us. If we continue in this attitude we deserve to lose our freedom and to become the slaves of a bureaucratic government. We have too much law talking and too little real law doing.

Mr. Alvord has attempted to simplify his discussion by laying down certain definitions of the terms he uses in his paper. Much of our talking at cross-purposes undoubtedly comes from the loose and incorrect use of terms. The speaker must emphatically dissent from Mr. Alvord in his definition of "reproduction cost", as he cannot see anything other than a begging of the question when the author says of this term, in parentheses, "Used in this paper generally as a short term for reproduction less depreciation". Here, through a definition of his own making, the author attempts to dispose of what is probably the most difficult question involved in the valuation problem.

When it is claimed that, in valuing plant for rate-making purposes, "Depreciation" should be deducted from the cost new of plant, what is meant by the term "Depreciation"?

If we may judge by the testimony, papers, and discussions involving this question, there is great confusion of thought as to the meaning of this term often so improperly used; but, if we follow to the end the statements and arguments of those who favor this deduction, we find that this term, "Depreciation," must be taken to cover, in connection with rate-making valuations, the accrued and to accrue liability for the future cost of renewing and replacing certain parts of plant which are to be retired from service due to any cause—physical decay, obsolescence, or inadequacy—these parts to be retired just as soon as they are no longer able to render effective and economical service, and not until then. These renewal and replacement costs are to be independent of the current repairs and minor renewals made

year by year, which are necessary to keep the plant up to the required efficiency pending these deferred major renewals and replacements. Mr. Humphreys.

It follows that such parts as are to be included in the estimate of accruing liability for renewals and replacements are those of longer life, and are not covered by the ordinary annual expenditures for upkeep; hence the estimate of annual charge against "Reserve" to meet this liability depends on the assumed effective life of the several parts and the present age as found.

We cannot fairly expect that such an estimate can be absolutely accurate. We cannot safely use, in our assumptions as to life, the data to be obtained from textbooks and the like. These tables of life expectation are supposed to be made up of averages from actual experience. If so, they certainly cannot fairly be applied in specific cases. The variations which are included to produce the average will, of course, be found also in the specific cases to which these average figures are applied; but, too frequently, these tables are only the expression of opinion of some "expert", competent or otherwise.

In any case, it is indefensible to use the life of even a certain standard piece of machinery or apparatus and apply that life to the case of another piece of exactly the same class, and even from the same manufacturer; for those of us who are familiar with operating conditions know full well how the differences in the management and upkeep of plant affect its length of useful service.

A still more reprehensible method, followed by some of those who use stated life figures as if they were proper for general application, is to quote from the report or testimony in a specific case, which figures are shown by the context to be adjusted to the specific case by considering the local and other controlling conditions. The speaker has such cases in mind, some of them where the fault has been due to ignorance or inexperience, but some of them, he is sorry to say, due to improper motives. In the most flagrant case known to the speaker, the testimony accompanying the table, not only repeatedly states that the life table is adjusted to the particular case, but the facts stated demonstrate that to use this life table for general application would be indefensible.

One feature in this table is particularly noticeable, namely, the non-inclusion of some parts having a total serviceable life of about 10 years, and this because, under the local conditions found, these parts were being renewed each year to the amount, in dollars, equal to about one-tenth of the total investment in these parts. Under other conditions, this item might carry the heaviest percentage for renewal of all the items in the table.

This exemplifies a point which has led to much talking at cross-purposes. If the renewals and replacements of all kinds fall in for renewal or replacement so that the cost thereof is found to be fairly

Mr. Humphreys.

uniform, and if, by expert consideration of all the facts, it is believed that each year is thus bearing its share of the total maintenance cost of the plant as a whole, then there is no need and no warrant for continuing an annual allowance to the depreciation reserve for periodic and final renewals or replacements.

This is a condition which may develop as years go on in the operation of the property. If the total annual expenditures for renewals and replacements do become practically uniform, from then on no annual contribution to the "depreciation" reserve need be set aside for renewals and replacements. However, we still have the liability for the period prior to the time when this condition of uniform annual renewal expenditures developed. If the contributions to the reserve had been correctly estimated and computed on the sinking-fund basis, then the interest on this reserve balance would be continually required for its portion of the annual renewals and replacements. Here it is to be borne in mind that in practice there cannot be, in the case of a growing property, a complete cycle. A complete cycle would require that it be measured by the part of plant of longest life, which must be a multiple of the lives of all the other parts. In practice, even if the plant were not growing, there could hardly be any such condition; and as a result, at the end of the longest life, there would be an overlap of liability for those parts having lives which were not factors of the longest life. Then, in this case, there will be, at the end of the so-called cycle, a liability for the renewals shown by this overlap, and there should be in the reserve an amount equal to this overlap of liability. The interest on this balance to the credit of the reserve would be part of the contribution to the cost of annual renewals. In such a case, this balance in the reserve account is necessary.

This condition of uniformity of annual expenditure may well be the case with large railroad systems composed of units built at different times and spread over wide areas involving variation in operating conditions.

The same may be true of other large public utilities of varied character. As to certain items, it may be true, even in the case of smaller and closely located plants, as, for instance, the consumers' meters of a gas company, which are found to be tested, repaired, or condemned in accordance with a scheme which calls for a complete cycle in, say, 5 years.

Here this item of plant is being completely maintained by current expenditures, and hence there is no additional liability to charge to profit and loss and to credit to reserve.

Now, the speaker's contention is that if the plant is maintained so as to render to the public thoroughly effective and economical service, there is no just reason why a deduction should be made because the time has not yet arrived for the final renewal or replacement of certain

Mr. Humphreys.

of the parts, for the renewal of which the corporation is liable and will be held liable by the public service commission. Whether the plant as a whole is maintained by current annual expenditures alone, or by the current ordinary expenditures plus the expenditures for deferred renewals covered by a reserve maintained by annual contributions, does not affect this question of deduction from plant value for depreciation. If the plant is maintained as well as it is possible to maintain it, and if, as it ages in certain of its parts, it is made young again by the renewal of those parts, then the owners are doing all in their power to preserve their property, in the interest of themselves and the public. Under these circumstances, to deduct for the accrued liability for renewals—a liability which cannot be met until the time for economical renewal arrives—must be confiscatory.

Further, the speaker believes that the day will come when the U. S. Supreme Court will so decide. It certainly will do so when at last its members come to realize that the basis for "depreciation" deduction is the acknowledgment on the part of the utilities of their liability for periodic and final renewals, which acknowledgment does or should take the form of certain statements on their books of account which act to prevent over-statement of profits and the payment of dividends not earned, and also places the burden of renewals on the consumers benefited, instead of on their successors. A reserve carried for periodic and final renewals of plant is simply an accounting device in the interest of more accurate statements of earnings and better financial practices.

Finally, what the speaker is contending for is that, in the case of a property honestly and economically managed, including the maintenance of the plant at a high degree of efficiency, we should have the privilege, when necessary, of spreading the cost of deferred plant renewals and replacements over all the probable years of service of each part of the plant by means of certain accounting processes, so that we shall not deceive ourselves as to our annual profits and shall place the current burden for upkeep where it belongs; and also we should enjoy this privilege without affording an excuse to commissions and Courts to deduct this reserve accumulation from cost new of plant by reason of the supposed acknowledgment on our part of actual plant depreciation.

Mr. Dunham.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—With an able attorney, the writer once examined a simple machine that showed the effect of the direct expansion of a steel rod. There was no leverage, nor multiplication of parts, nor of motion. A weight was held in place by friction. If the rod expanded, the friction was lessened and the weight fell. If one merely put his fingers on the rod, the weight

* New York City.

Mr. Dunham.

went down. If he breathed once upon it, the same result followed. A slight force applied to the end of the heavy supporting cast-iron bed-plate warped the casting, and the weight descended.

The attorney said:

"This is the finest object lesson of a life time. Now it is known there can be nothing firm or rigid on our planet. Everything is flexible. The mountain ranges move away from the sun. Nothing is fixed."

At that moment, laws and Court decisions would have seemed to that attorney less permanent than they appear to be to the author of this illuminating and interesting paper. His masterly way of meeting the difficulties everywhere prevalent, and so well set out, will not suffer if attention be drawn to this one feature of assigning to Court decisions an unexpected degree of permanence. The paper itself is not only a protest, but, through its merits, may do much toward a change in legislation or opinions about which the author has taken a hopeless view.

In the discussion of another paper,* the writer referred to new rules from the Courts, holding that the rules are valuable, but should not be substitutes for vital individual work no less searching than that which they embody.

After all that has been written and said about methods and details in the determination of rates, there may be little patience with or space for new suggestions; and yet it seems that for each particular city there should be an appropriate valuation and a schedule depending on, not only all the local conditions of cost, construction, efficiency, permanence, etc., but on other known facts pertaining to that city, surrounding cities, and even the relation of these to the State. A schedule thus determined should be a schedule for that city, and should vary but little, whether the ownership were private or municipal. To show how easily an appraiser is led beyond the municipal boundary, it is only necessary to cite cases that are quite common in many parts of the country.

A city is located and established in such a way, with respect to any possible water supply, that great expense for installation and operation must be incurred. A valuation is based on these conditions. The rates naturally and comparatively would be high. In a near-by city quite opposite conditions exist. Excellent water can be obtained and purveyed with small expense and attention. Naturally, the schedule would be a low one. Resting each case on the same principles, and fixing valuation and rates in the same manner, it may be easily shown that one city would be supplied with water at a fraction of the cost in the other. The question to be met by appraisers,

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 373.

and by utility commissions as well, is whether other features than those pertaining definitely to the works in those cities should be considered. Where there is a little rivalry for manufacturing and other lines of development, it would be extremely difficult to prevent the common council from making sacrifices in some way through which their rates for water would be brought down to the level of those in the more favored community. Added weight is given to this view by the fact that, if the plant were owned by a private company, the same logic and necessity would have like weight and influence. There is no doubt that some modifications would follow, and the fact that this result would be reached in a business way by any group of business men furnishes sufficient evidence that attention must be given to the principles that lead to such action. The existence of the conditions in the second city would affect the valuation of the plant in the first city. These principles have their place, in common with any of the other facts on which valuation and rates may be based. That a commission would recognize them as important and legitimate can readily be inferred. Their action in the matter would be a recognition of business principles not unlike that which obtains in fixing differential rates in transportation. A fundamental reason or guide for their conclusions would be found in the fact that the two communities were a part of the larger commonwealth, and, in fairness, the best interests of the whole should not be neglected. Mr. Dunham.

The relation of a commission to natural conditions that affect unequally two near-by communities can be illustrated by assuming that through some oversight the rates for the city that could obtain a supply only at great expense were based on a 7% return, while 5% had been fixed for the more fortunate city. This would be followed by protests of greater force than could be expected, if the percentages of return had been reversed, further showing the existence of substantial reasons for introducing a wide equalization method. It might be difficult to say to what extent a commission has the right to introduce differential or compensating rates in adjacent cities, but the right should be better sustained than is the right of a commission to fix meter rates for water at a high figure in one city and at figures below the cost of pumping in a similar city obtaining its supply from similar sources and under like construction expense.

The refusal in some States to permit further bestowal of free water can only prove beneficial to all, but it should not be forgotten that to introduce rates below actual cost is not unlike a return to free water privileges against which the same authority enters protest.

The responsibility of a city for its own part in the design and construction of a company utility is not often referred to, but this affects valuation. Mistakes at the beginning, or at a later period, usually involve both city and company. The city's approval is required

Mr. Dunham. in connection with the design and on the completion of the works. Extensions are usually defined and ordered by the city. If it becomes apparent that errors have been made and funds expended that fail to bring proper returns, it can hardly be claimed that only one of the contracting parties should suffer. In a recent case, a city ordered an expensive intake pipe from a river for fire protection; then a city sewer system was constructed in such a way that the intake pipe could not be used; then, under a purchase clause, the city obtained the plant, demanding that the cost of the intake pipe should not be included, as it had no present value. This demand was refused by the State commission. A growing spirit of fairness on both sides lessens many of the difficulties in utility affairs.

Mr. Knox. Stuart K. Knox,* M. Am. Soc. C. E. (by letter).—The one ground for really serious difference of opinion in Mr. Alvord's otherwise admirable review of the Fundamental Principles of Public Utility Valuation, will be found by most engineers in the author's treatment of the subject of depreciation in its relation to the value on which rates should be adjusted to yield a fair return.

The writer infers from this and other recent writings by the same author that Mr. Alvord is still of the opinion that the deduction of physical depreciation from value new in rate-making appraisals is morally justifiable and legally necessary, even when the so-called annual depreciation reserves are computed on the sinking-fund basis.

This idea, still widely held, appears to be based largely on two assumptions, both false, as follows:

(*a*) That the deduction is morally justifiable because appreciations, in the form of unearned increments, are, perforce, credited to the owner.

(*b*) That the deduction is legally necessary because of the existence of some fundamental principle of law which has compelled the Courts thus to decide.

Thus Mr. Alvord, in closing the discussion of his former paper, "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return,"† says:

"Objectors [to the deduction of physical depreciation where reserves are computed on the sinking-fund basis] do not take note of the fact that the appreciations of plants and property are included by the reproduction method, and must be considered in connection with depreciations and that, if the objectors would exclude depreciation, they must logically exclude the appreciation that the reproduction

* Montclair, N. J.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

method usually introduces. The same arguments which one would use to exclude depreciation can be profitably used to show that questions of appreciation should not be entertained." Mr. Knox.

Thus, also, in the present paper, he lays stress on the fact that the Supreme Court of the United States, under compulsion of the Constitution, which secures an owner in the possession of unearned increments, has held: "that the value of the property is to be determined as of the time when the inquiry is made regarding the rates", and clearly implies his (erroneous) belief that the deduction of physical depreciation is in all cases necessary in ascertaining the value of a property "now".

The assumptions, (*a*) and (*b*), having been characterized as false, it remains to show wherein and to what extent they are so.

Assumption (*a*) is false because physical depreciation is not the antithesis of any appreciation which can occur. There is no method of valuation now in use which credits an owner with appreciations in value without automatically debiting him with their antithetic depreciations. Thus the antithesis of appreciation in the value of real estate is depreciation in its value; and the application of the reproduction cost method, which credits the owner with one, debits him with the other. The same is true of appreciations in the value of labor and materials, which may have occurred since the date of original construction, of appreciation due to the placing of pavement over pipe, and of other increments which are the result of what might be called external causes. Each has its antithetic depreciation, and the method which allows for one takes account of the other. Not even the advocates of original cost to date as the proper basis for rate-making advise depriving the owner of unearned increments without, at the same time, saving him harmless from unmerited decrements in the value of his property.

The foregoing facts were recently emphasized, as the writer now recalls, by George F. Swain, Past-President, Am. Soc. C. E., who pointed out that a sharp distinction should be drawn between physical depreciation resulting from use and the shrinkage in value which may be caused by recessions in the market value of real estate, the discovery or invention of cheaper and more efficient methods of manufacture and construction, and the like.

The wide-spread failure to perceive, or inability to realize the important and real differences between these fundamentally dissimilar things appears to be due in no small measure to the indiscriminate and unfortunate use of the single word "depreciation" to designate both of them. (See Mr. Alvord's table on page 2403*). "Deferred maintenance", "Impending replacements", or some similar designation

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Knox. would describe more aptly and accurately what is meant when physical depreciation is spoken of, than does the word "depreciation".

The term "deferred maintenance", thus used, for example, might be defined as Mr. Alvord has defined "depreciation", page 2392,* as "the lessened [decrease in] value of any property, structure, or machine, due either to its wear, loss of usefulness, growing lack of adaptation, or approaching abandonment", or it might be defined otherwise, as "that which gives rise to the necessity for eventual replacement", or as "wastage or consumption of physical plant through use, to offset which so-called depreciation reserves are permitted to be made from earnings, before computing net income".

"Deferred maintenance" does not connote the antithesis of an unearned increment. It does connote that steady and inevitable wearing out of plant and equipment, the entire cost of which is as much a part of operating expense as is the cost of coal. It connotes, also, the close analogy which exists between the thing described and current maintenance, from which it differs materially only in respect of the magnitude, and evenness in annual distribution of the replacements required, and with which it merges completely when this distribution is naturally uniform.

Every intelligent investor who places money in public utility or other property, does so with the full understanding that it will be subject to the normal vicissitudes which beset invested capital everywhere. The investor expects to be, and is, debited with all losses resulting from bad judgment, miscalculation of future growth, and the various unforeseen and unforeseeable contingencies which cause the value of invested capital to fluctuate. No one, for example, ever heard of a fund being amortized out of earnings to offset a shrinkage in value due to the bursting of a real estate boom. In consideration of the assumption of these risks, every investor expects to reap the benefit of his own good judgment and foresight. He rightly expects to be credited even with unforeseen and unearned increments with which fortune may favor him, and these increments in value are in fact vested in him by the Constitution of the United States, as interpreted by the Supreme Court.

Physical depreciation is not such a vicissitude as is above described. It is neither unforeseeable nor unforeseen, but is an operating cost the occurrence of which is as certain, and the amount of which may be predicted with as much confidence, as future taxes. This principle has been fully recognized by the Courts, including the Supreme Court, and this fact brings us to the consideration of what the attitude of the Courts actually has been with regard to this matter of depreciation in its relation to value, and to the examination of Assumption (*b*), characterized as false, "That the deduction is legally necessary

* *Loc. cit.*

because of the existence of some fundamental principle of law which has compelled the Courts thus to decide". Mr. Knox.

The writer does not recall an instance in which the Courts have decided with specific reference to the situation where reserves have been, and will continue to be, computed on the sinking-fund basis, that physical depreciation must be deducted from value new in rate-making appraisals. Certainly no such ruling is discernible in any of the decisions quoted by Mr. Alvord. The reader searches these quotations in vain for any reference to the method to be used in computing the annual reserve, whether sinking fund or otherwise, and this is the crux of the whole matter.

So far as the writer knows, it is not seriously contended by any valuation expert that the owner will suffer financial loss, if, where reserves are computed by the straight-line method, physical depreciation is deducted from value new in ascertaining the value on which the owner should be allowed to earn a fair return. As was demonstrated at length in the discussion of Mr. Alvord's previous paper, the value on which the owner should be allowed to earn a fair return depends wholly on the provisions made to offset physical depreciation, and on whether or not these provisions effect a virtual refund of capital. If they do effect a refund of capital, then it is equitable, in rate-making appraisals, to deduct from value new the amount of the capital thus refunded. If the amount thus refunded coincides with the physical depreciation, it is equitable to deduct the physical depreciation, and this is true whether or not the refunded capital is kept in bank as an actual tangible depreciation fund.

The writer has stated that he recalls no instance in which the Courts have decided with specific reference to the situation where reserves have been, and will continue to be, computed on the sinking-fund basis, that physical depreciation must be deducted from value new in rate-making appraisals, and it is certain that the Courts could not thus decide without involving themselves in a fatal inconsistency.

The Supreme Court of the United States, in the Knoxville case, ruled that:

"The Company is not bound to see its property gradually waste without making provision out of earnings for its replacement. It is entitled to see that from earnings the value of the property involved is kept unimpaired, so that, at the end of any given term of years, the investment [so far as the effects of this wasting is concerned] remains as it was at the beginning."

If this means anything of value to the owner, it means that the Courts will not suffer him, against his will, to be subjected to loss solely as a result of the inevitable physical depreciation of his plant.

Mr. Knox. Now it is a fact, susceptible of mathematical proof, that an owner whose annual reserves are sufficient only to amortize the sums required for replacement, when credited with interest accretions, *i. e.*, whose annual reserves are correctly computed on the sinking-fund basis, does suffer a loss from no other cause than the wasting of his property through use, unless rates are adjusted to yield, in addition to operating expenses and this annual reserve, a fair return on "value new". For if his property suffers no physical depreciation, his profits represent a fair return on "value new"; whereas, if it does suffer physical depreciation, his profits represent a fair return on an amount less than "value new".

Therefore, under these circumstances, if a Court rules as the Supreme Court did in the Knoxville case, and at the same time holds that rates must be adjusted to yield a fair return only on value new less physical depreciation, it is obvious that it has decided two interrelated and interdependent matters in inconsistent and irreconcilable ways, and that the question confronting the engineer, in the premises, is not whether he shall cut his valuation to fit Court rulings, but by which of two conflicting decisions he shall be governed. The presumption is strong that the Courts have not, either knowingly or unintentionally, placed themselves in so anomalous and untenable a position.

In Mr. Alvord's language, the thing which is valued is a property. The Courts have said that it is the property which is used and useful to the public; but a reserve fund which is uninterruptedly used in amortizing and supplying the sums needed for necessary replacements is as much part of the used and useful property as is a pumping engine or a locomotive. Few engineers would question this if the fund were a tangible one actually maintained in bank, and as soon as one admits this fact he is committed to the proposition that any fund thus computed, regardless of its disposition, is a part of the used and useful property. For if the owner diverts the annual reserves to other uses, the fund will not be available when required for replacements, and these, therefore, must be paid for out of the owner's pocket. Eventually, he (or the stock and bondhholders) will be obliged to supply, from his own resources, not only the annual sums diverted, but also the accumulated compound interest on these amounts. The net effect of the operation, therefore, is that the company has loaned the depreciation reserves to the owner (or to its stockholders) "on call" at the rate of interest at which the fund was computed, and that the entire fund with estimated interest accretions remains part of the used and useful property as much as if it were actually in bank. There is nothing in the principle, that the used and useful property is the thing to be valued, which would act as a bar to the inclusion as value of the accrued physical depreciation.

Mr. Knox

The Courts have also said (and Mr. Alvord makes a great point of this) that it is the value of the property as of the date of the appraisal which is to be ascertained. Although there is discernible in the decisions of the lower Courts the same misapprehension which leads Mr. Alvord to confuse physical depreciation with ordinary shrinkage in value, one cannot but be impressed with the fact, in reading these decisions, that the real rock on which the Courts have taken their stand is that it is the value of the property "now" which must be ascertained. The reasoning by which the Courts have reached this conclusion is as clear as noonday, and the logic quite unassailable. The Constitution secures the owner in the possession of unearned increments. It follows logically that he should be obliged to bear losses resulting from antithetic decrements. In applying the reproduction-cost method, it is the cost of reproducing the property "as of to-day", using average current prices for labor and materials, which must be ascertained, and not the cost of reproducing the property under the conditions which existed at the time it was constructed. In deciding this point, the Courts are on accustomed ground. It is undoubtedly true that logic has compelled them to decide that the value to be ascertained is the value "now"; but the conclusion that physical depreciation must always be deducted in arriving at the value now is demonstrably a *non-sequitur*.

If it be acknowledged that a given property at a given time has an intrinsic or fair "value new", which may be ascertained independently of a consideration of rates (and the whole theory of the valuation of rate-controlled public utilities is predicated on this assumption), and if it be admitted that the accrued physical depreciation on a given property at a given time is a definite quantity, the value of which may be ascertained with reasonable accuracy, then we are committed to the proposition that "value new" and "value now" are synonymous expressions, so long as the so-called depreciation reserves are computed on the sinking-fund basis.

For, let V_N = value new, D = accrued physical depreciation, and V_P property value, or value new less physical depreciation. Then we may write the expression:

$$V_N = V_P + D,$$

in which V_N, D, and hence V_P, are definite quantities which may, by hypothesis, be ascertained independently of a consideration of rates, for any given property at a given time.

Now the "value as of the date of the appraisal" or "value now" of a property, the average life expectancy of which has been diminished as a result of use, is represented by and equal to the total capital investment which a willing purchaser must make to acquire it from a seller who is willing, but not compelled, to sell.

Mr. Knox.

To acquire a property on which there exists an accrued physical depreciation, D, a purchaser, A, would be obliged, and could afford to pay to the owner, B, a sum equal to its deteriorated property value, V_P, as V_P is an intrinsic value ascertained independently of a consideration of rates, and as such must, of course, be sustained by the rates; but, if accrued physical depreciation and future depreciation reserves are to be computed on the sinking-fund basis, V_P will not represent the total investment necessary to acquire the property, and hence will not represent value now. If the purchaser, A, will be allowed to reserve annually from earnings only such sums as will with interest accretions amortize the value new of each plant element, in a time interval equal to its age plus its remaining life expectancy (and this is the correct basis for computation of a proper sinking-fund reserve), then the total investment necessary to acquire the property will be $V_P + D$. For if the purchaser, A, is to place himself in a position to make replacements as these become necessary, without the investment of new capital, he must on the date of the purchase, set aside at interest an amount, D, equal to the physical depreciation on the plant. If he does not, he has no less effectively committed himself, because, in taking over an old property in exchange for a sum, V_P, equal to its intrinsic value, he has also by hypothesis assumed the obligation at some future time to invest in replacements additional new capital having a present value of D.

Now, as things equal to the same thing are equal to each other, it follows that:

Value new $= V_P + D =$ value now, and that when depreciation reserves are computed correctly on the sinking-fund basis, "Value now" and "Value new" are synonymous expressions, which is what it was desired to prove.

Thus the Courts are correct in ruling that "value now" is the proper value to use as a basis for rates, and incorrect, if at all, only in failing to perceive or recognize that the question of whether or not physical depreciation shall be deducted from "value new" depends entirely on the provisions made to take care of future replacements, and on whether or not these provisions are such as virtually to effect a refund of the capital wasted through use.

The writer has already, in his discussion of Mr. Alvord's former paper, reasoned at length with regard to the relationship between the various methods of providing for past and future physical depreciation, on the effect of the several methods on fair rates, and on the values which serve as their basis. He will not repeat that discussion here except to say that no one of the conclusions there reached is vitiated by Mr. Alvord's just criticism that it was written primarily with reference to cases in which Value New, Reproduction Cost New,

and Total Cost to Date were synonymous expressions. "Fundamental Principles of Public Utility Valuation", the title of Mr. Alvord's present paper, must be sufficiently broad in scope to cover all cases which may arise, and cases frequently do arise in which value new practically coincides, not only with reproduction cost new, but in recently constructed properties with total cost to date, as well. Mr. Knox.

C. E. Grunsky,* M. Am. Soc. C. E. (by letter).—Although he does not agree with the author in his conclusions, which are based on the assumption that, because the Supreme Court and other Courts have reached a decision, certain fundamental principles are finally settled, the writer desires to compliment him on his attempt to establish such principles. There is need at the present time for careful consideration, and much good should come from the discussion of the question, now squarely presented: Must the opinion of the United States Supreme Court be accepted as final, even though it be an economic fallacy? Perhaps such opinion may be in conformity with the law. If economically wrong, should not the law be changed? Mr. Grunsky.

The net profits and, consequently, the value of any property, such as a public utility, other conditions remaining the same, depend on, and fluctuate with, the rates. The rates determine value, and therefore the converse, that value should be the guide in fixing rates, is fundamentally wrong. It is the height of absurdity for the law, the Courts, or public utility commissions, to make value—however defined or restricted—a basis for the fixing of rates.

The uselessness of thus moving in a circle—which follows when value is considered in establishing rates—can, when properly pointed out, be grasped by the dullest mind, and is gradually being recognized by those who are concerned with the establishment of rates.

If the law is at variance with this fundamental truth, the law should be changed. Where the interpretation of the law by the Courts is at variance with this fundamental truth, this interpretation should be modified. Engineers and economists should not accept supinely the erroneous conclusions of the Courts, but should continue the campaign of education until the true principles are recognized and firmly established.

The writer, therefore, cannot agree with the author's conclusions which are based on the premise that, because the Courts have said so, some method must be found to establish fair value of the public utility under the assumption that earnings may be regarded as dependent on, instead of as the cause of, value.

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* San Francisco, Cal.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE VALUATION OF PUBLIC UTILITY PROPERTY.

Discussion.*

By Messrs. Charles Rufus Harte, and F. W. Green.

Mr. Harte.

Charles Rufus Harte,† M. Am. Soc. C. E. (by letter).—In endeavoring to establish a foundation for his proposition that "in any valuation or appraisal that is to be made for purposes of rate-making or security issues, the proper figure to be used is the actual cost to date of the work in question", Mr. Gandolfo points out "differences" between the private business, the private corporation, and the public utility corporation, which, at least to the writer, seem of degree rather than of kind.

Proportionately, the private business deals as extensively with that portion of the public in its field as either of the other groups; its commercial paper stands on the value of the business, including the exceedingly important intangibles of good will and business reputation, precisely as the securities of the others stand or fall—a glance at the proceedings of any Court of Bankruptcy will show how often misjudgment of these factors has caused failure, and the "compositions" effected testify to the intangibility of much, if not all, of the assets; the liability is proportional to the holding in any case—the sole owner, whether he represents a private business or 100% of corporation security, is liable in full; and the frequency with which private business expansions are made on personal or firm notes which lack support and are not "paid back from the profits of the additional business before any benefit" is "expected by the owner" keeps many a lawyer busy.

No small part of the failures which not infrequently occur when old and apparently prosperous concerns attempt to expand, is due

* Continued from December, 1914, *Proceedings*.

† New Haven, Conn.

Mr. Harte.

directly to bookkeeping which has figured profit on an original cost instead of the greatly increased value of the plant, and predicated the expected return from the expansion on that basis.

That "the private concern never asks any favors of the municipality or of the State", is a statement hardly borne out by the facts. Much the larger portion of the occupancy of the highway is by the teams of individuals, no less by favor of the municipality than the use for tracks, wires, pipes, or the like, of the public utility corporation; and no small part of the time of the common council or its equivalent is taken up with the consideration of petitions for marquee, area-way, connecting bridge, or wire and pipe location rights by individuals.

It is true that the private concern cannot "invoke the right of the law of eminent domain", but, for that matter, neither can a large proportion of the public utilities. That right is not, as the author seems to intimate, a special privilege to a public utility "to further its own affairs", but, on the contrary, is a State prerogative to protect the public against the individual who, otherwise, might block needed improvement; it is delegated only to further that end, and, in its exercise, the public necessity must always be specifically shown.

The reasons given for organizing a private corporation are equally cogent in the foundation of a partnership, which is nothing more than a corporation of special form; indeed, the individual owner of a business is the limiting case of the corporation.

The most important reason of all for incorporating is only half touched on; modern business demands such large capital that few care to put so large a part of their money in one investment, but would rather divide it among many projects, in order to reduce the extent of the loss in case of failure of one or more. Such scattering of investments, however, does not relieve the investor of one whit of his responsibility, which is exactly in proportion to his holding.

The "essential differences", as summarized by the author, hardly seem to be such. All the real owners of the utility may not control, but the majority do; the minority are in the same position as the minority in any republican form of government, and the aggrieved stockholder has the same right to invoke the protection of the Courts—and uses it, too—as has the taxpayer who is dissatisfied with the action of his governmental body.

The other three "principles" are as characteristic of private business as of corporations, whether private or public.

With the author's general idea of a public utility, that it does a business essential to the public welfare and from its nature—the great plant required, the physical exclusion of a competitor, the necessity of centralized control, etc.—is essentially a monopoly, there can be little dissent, but its difference from the other two groups is only in the fact that, monopolizing a public necessity, it is held

Mr. Harte.

subject to public regulation purely as a matter of protection, and not because of special privileges enjoyed; by far the greater part of the "Sherman Act" cases decided or under way are against businesses which by their expansion are alleged to have monopolized the market and thus to have become public utilities, although enjoying no rights other than those of a private business.

In his argument to the main question, Mr. Gandolfo states that the decisions in favor of "reproduction less depreciation" are based on three arguments:

1.—Inequity of depriving old companies of increased value in property;
2.—Impossibility, in the case of older companies, of determining cost to date; and
3.—The protection afforded by the Fourteenth Amendment.

A more correct statement would have been restricted to the third reason, together with the interpretation by the Courts of this Amendment. Until the Constitution is further amended, "property" is protected against confiscation without compensation; until another amendment, or the reversal of many and consistent decisions of the Supreme Court and the establishment of a new ruling, the value of property is its worth at the time in question. So firmly is this established as the "general rule" (and for the benefit of the author, who is apparently unfamiliar with this expression, it might be said that it is a common law term meaning the established judicial attitude toward the question at issue), that we find Justice Holmes saying:

"It is no longer open to dispute that under the Constitution what the Company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used by the public." (San Diego Land and Town Co. *vs.* Jasper, 189 U. S., 439, page 442.)

The "actual cost to date" method fails to give recognition to that which the engineer should be particularly jealous to safeguard, for it is that alone which warrants his professional existence—scientific knowledge.

The intelligence that forsees the future industrial center in the wilderness of the present; the knowledge and courage that in the face of all but insuperable difficulties carry through to successful completion a great development—these which are really the chief elements in creating the real value of the utility, are entirely ignored by the "cost to date" method, which, by accepting the wasteful and unwarranted expenditures of the dreamer and of him whose daring is that of ignorance because the investment was made in misguided honesty, puts a premium on incompetency and extravagance.

Mr. Harte.

In the author's assumption, on page 2457,* he apparently confuses property and cost. The property is measured by the value, and not by the cost, of what is held, and it is the value, not the cost, to which the protection of the Courts is extended. The position of the Courts is very clearly set out by Justice Hughes:

"It is clear that in ascertaining the present value we are not limited to the consideration of the amount of the actual investment. If that has been reckless or improvident, losses may be sustained which the community does not underwrite. As the company may not be protected in its actual investment, if the value of its property be plainly less, so the making of a just return for the use of the property involves the recognition of its fair value if it be more than its cost." (The Minnesota Rate Cases, page 46, Senate Doc. No. 54, 63d Cong., 230 U. S., 352.)

The reason for this is well founded. The public, in constructing its own and competing plant, would have to pay the prices as of the date of work; it would enjoy the benefit of advances in the art; it would have to bear the burden of increased costs over those of an earlier time. It is obviously as unjust to compel the public to pay rates on a cost in excess of what it would have to pay to reproduce the facilities as it is to compel the utility to make rates on a cost less than that which would be required if the public either bought the old plant at a fair value, or built a new one.

To use the original cost of the Tring Cutting—aside from the fact that, unless there is much more and better data than that of "an old print," any estimate of the original cost would be far more speculative than one based on modern methods—would deprive the public of the benefit of progress in excavation apparatus and methods; to hold the early-built railroads to the very low cost of trestling built from timber cut along the line would deprive them of an element of value, in many cases actually paid in by the company through renewals of the old structure with new materials at the higher later price, which, affording no betterment, must be charged to operation.

The author refers to the holdback as tending to reduce the interest charge. As a matter of rigid accuracy this should be considered, but the actual amount involved is small, with the entire project covered by contract, and with the construction funds drawing full interest up to the time of their use, there would be a saving, on one-tenth of the cost, of one-sixth of the rate of interest. Usually, however, the company purchases the bulk of the material, while the construction fund, because of its activity, rarely commands one-third of the ruling interest, and often less than this, making the net saving due to the holdback about one-fiftieth of 1 per cent.

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Harte.

The writer is not in accord with many of Mr. Gandolfo's propositions, but his treatment of depreciation and its allies, in the clear recognition of the distinction between accounting depreciation and actual depreciation, seems to be one of the few rays of bright light in the fog in which so many writers have concealed the facts.

It is in the consideration of "Valuation for Purchase or Sale" that, in the writer's judgment, the error of the author's ways becomes obvious.

Let us consider a property the returns of which have been fixed by the cost to date, but having, by virtue of wise handling, a value 50% greater than its cost. B buys the property at its value; can there be any question as to his absolute right to jump the rates 50%, the cost to date having so jumped without one whit of change in service?

Or, if the contrary condition obtained, B buying for 50% less than cost, could he prevent a rate reduction of 50% although he gave exactly the same service as his predecessor?

The fact is, that, as the author himself says, "With our rapid development, the purely private business of to-day may become the public utility of to-morrow." There is no hard and fast line of distinction, either between the two classes of business, or between the principles and practices applying to them.

Unless the owner be permitted to earn at least a fair return on the capital which his investment represents—and that is obviously what it would bring in a free sale—there is confiscation; to base rates on cost to date, when the investment in a free sale would bring much less, is unfair to the rate-payer; it would seem that the Supreme Court, in its consistent stand for value to date, had logic with it.

Mr. Green.

F. W. Green,* Assoc. M. Am. Soc. C. E. (by letter).—Valuation is a subject which is rather "meat for strong men", than "milk for babes". To be an expert, one must not only have the ability to use the yard-stick of the engineer, but that of the lawyer, the publicist, and the economist, as well. The subject looms larger, perhaps, in its economic aspect than in any other. Any valuation, or system of valuation, based on principles which fail to satisfy the requirements of law, of economics, and of engineering, is, *ipso facto,* to be rejected.

It is but natural that there should be divergent views on a matter so involved. In writing about it, we are all apt to reflect unconsciously our hobbies and our prejudices, and these are often as likely to be wrong, as to be right. Although the writer cannot agree with the author in many of his conclusions, yet he believes the paper offers many opportunities for profitable discussion, and elucidates and clarifies the subject in many particulars.

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* Stamps, Ark.

Mr. Green.

In all the discussion regarding valuation, the writer fails to recall an instance in which attention has been called to the radical difference between electric, gas, and water utilities, on the one hand, and steam railroads on the other. When the entire output of a utility is one thing—water, gas, or electric current—the units of service are homogeneous and commensurable. If we seek fair rates for service to the consumer, we may write the equation:

Gross Earnings minus the sum of Operating Expenses and Taxes, Reserves for Depreciation, and Fixed Charges, equals Fair Rate of Return multiplied by Fair Value.

From this we may write:

Fair Rate for Service equals the product of Fair Rate of Return and Fair Value plus the sum of Operating Expenses and Taxes, Depreciation Reserves, and Fixed Charges, divided by Units of Service.

Per contra, the units of service produced by the steam railroad are neither homogeneous nor commensurable; and unless we are prepared to say that a ton-mile shall be a ton-mile, whether the commodity be suburban passengers, immigrants, iron ore, straw hats, coal, or silverware, either car-load or less than car-load, and that a uniform rate per unit of service shall apply uniformly and invariably, it would seem to the writer that some of the theories of valuation as applied to common carriers must fail. Valuation is not herein deprecated; it is believed that it will do more to dispel the misapprehension and prejudice, which have been created and fostered by certain persons with political ambitions, than any other single cause; but, at the same time, it would be well to point out this condition now, lest some of us be disappointed hereafter.

On pages 2444-2445* the author betrays a mistaken attitude as to eminent domain. He states: "Nor can it [the private concern] invoke the right of the law of eminent domain to further its own affairs." History teaches the error of this concept. Eminent domain, the prerogative of Government, exercised only in and for the public interest, was delegated first to certain canal companies, and later to railway corporations. In the delegation of this power, the Government of every State, as far as the writer can learn, specifically restricted its exercise to purposes in which there were real and vital public interests, and these had to be proven before the right could be exercised. A number of cases may be referred to in any good law library, in which it has been held that the right of eminent domain may not be exercised without restriction, or merely to serve a private interest, for instance, for a track to a private industry, as a rock quarry, manufacturing plant, etc. When such right is exercised by a carrier, it is not by virtue of any inherent right vested in the carrier *per se,* but rather because it is more expeditious and convenient for the carrier as the

* *Proceedings,* Am. Soc. C. E., for October, 1914.

agent *ad hoc* of the Government to exercise the right, always and only, for the benefit of the public and with ample protection of the rights of the private citizen whose land has thus been appropriated to public use. Otherwise, if there were no such delegation of governmental authority to carriers authorizing them to use, within prescribed limits, the governmental right of eminent domain, the Government itself would have to exercise the right, and, later, after having acquired the land expropriated, turn it over to the carrier. Mr. Green.

If one will but take the time to examine the numerous citations pertinent, in a judicial frame of mind and without partisan prejudice, this view finds confirmation in many various phases. For instance, in Louisiana, a case is on record in which it was held that one road could condemn a right of way over and along the right of way of another road on the grounds that although the old road owned the land and was using it for right of way, yet it was to the public interest that the new road be not obstructed; and as the old road had been permitted the exercise of the governmental right of eminent domain, in the interest of the public, it could not use the benefits acquired thus, to obstruct the new road in the exercise of a similar right; and all because *salus populi suprema lex est.*

As to the author's reference to watered stock (page 2448*) it seems to the writer that this is really nothing more than faith—"the substance of things hoped for, the essence of things unseen." If a farmer acquires land for $10 000, and refuses to sell it for less than $20 000, has he not watered his value 100 per cent.? But, does he get any more per bushel for the corn raised on this land?

If banks are not well managed, the remedy is to change management when bad loans are made on inadequate security (page 2448*). And (page 2449*) the doctrine of *caveat emptor* applies in the purchase of railway securities, as fully as in all other human activities. The Government which, with a noble compassion, and an attitude of exalted altruism, should undertake to protect fools from their folly, would have a bigger job than the Panama Canal.

Our friends who devoutly espouse Socialism, believe in the public control and management of all the utilities of production. To the writer it seems that the author's definition (page 2450*) of a public utility, would cover a bakery, a haberdashery, or a movie; for what are more vital to the community than bread, clothing, and recreation? Although it is necessary, of course, that certain public utilities be regulated in the public interest, it would seem equally necessary that we do not permit ourselves to drift into the conception that we should have one kind of law for certain enterprises, and another kind for others.

* *Loc. cit.*

Mr. Green.

The author's questions (page 2451*) as to what items should be taken into consideration are answered in the first sentence of the succeeding paragraph; but the answer is apparently disregarded in his advocacy of the principle of actual cost to date. The writer cannot agree with the author (page 2452*) that "The object of the valuation very properly should affect, not only the final figure, but possibly the method of conducting the examination as well." If a valuation is real, it should be the same for rates, taxes, issuances of securities, purchase, or sale. Otherwise we should have the ludicrous condition of a high valuation for taxation, and a low valuation for rates, as was the case, a few years ago, in Texas.

The argument against the allowance of "unearned increment" seems to be inconsistent. In the case of a company deferring the purchase of land, say, 20 years, until needed, the author would allow "actual cost"; but if the land were acquired by the company at the beginning of the 20-year period for, say, 10% of the amount paid at the end of that period, the author would allow only "actual cost". In the first instance, he would pay the "unearned increment" by including it in his valuation; in the second case, he would not allow it. From the standpoint of the rate-payer, why should he pay more in one case than in the other? If the "unearned increment" is justly due to the virtuous citizen, so also to the vituperated corporation.

On page 2462,* the author criticizes roads which now find it necessary to adjust their plant to present traffic, by various corrections in grade and alignment; but, in the second following paragraph, he would not include "excess size" of plant in his valuation. How such treatment of the subject could be called a "valuation" is hard to conceive. A corporation that would continue as such, if thus treated, would certainly be a wonderful thing to contemplate.

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* *Loc. cit.*

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PAPERS AND DISCUSSIONS

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INVESTIGATION OF THE PERFORMANCE OF A REACTION TURBINE.

Discussion.*

By Messrs. S. J. Zowski and H. Birchard Taylor.

Mr. Zowski.

S. J. Zowski,† Esq. (by letter).—Regarding the question of head for which a water turbine is responsible: In Paragraph 5 the author gives a formula which, as a close examination will show, defines the turbine head as the vertical distance from the level in the forebay, at the entry into the pipe line, to that in the tail-race, around the draft-tube, minus the friction head of the pipe line, minus the friction head in the draft-tube, minus the final discharge velocity head.

In Paragraph 6 he gives a formula which defines the turbine head as the same vertical distance between the two levels, minus the friction head in the pipe line.

This latter definition of the net head is that which has been used almost invariably, and the writer does not see any sufficient reason for abandoning it, at least, not when the definition of head serves the purpose of settling the question of efficiency guaranties.

The first definition eliminates the friction and final discharge loss in the draft-tube, thus not charging these losses to the turbine. This is an unwarranted—if not directly incorrect—method of procedure. The draft-tube is an essential part of the water turbine, its function being to make good in what the runner fails and frequently must fail. It utilizes or recovers that amount of energy which the runner does not, or, owing to some necessary limitations in its design, cannot utilize. With some wheels this amount is necessarily relatively large, so that the efficiencies of these turbines would not have accept-

* This discussion (of the paper by R. L. Daugherty, Esq., published in October, 1914, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. of Mech. Eng., Univ. of Michigan, Ann Arbor, Mich.

Mr. Zowski.

able values were it not for the use of a well-proportioned draft-tube. The draft-tube, therefore, is essential as a part of the prime mover and, consequently, whatever losses are obtained therein must be charged to the whole turbine. This explains why turbine builders and designers always insist that the draft-tube be either designed by them, or, at least, that its design, if made by others, shall be approved by them.

The elimination of the draft-tube loss would be parallel to the elimination of the area below the exhaust line of an indicator card of a steam engine. Nobody would approve of that, when the efficiency of the engine is determined from the indicator cards. Only in one instance might there be reason for the elimination of draft-tube losses, namely, when comparisons are made between turbines, which have had to receive draft-tubes of widely differing proportions.

When, however, the matter of efficiency guaranty is being decided or agreed on, there is no excuse for this elimination. The designer and builder, if he sees that local conditions over which he has no control will not allow him to put in a draft-tube of the proportions which he would like to use, will take this into account accordingly, and make an allowance in his guaranty.

Mr. Taylor.

H. Birchard Taylor,* Esq. (by letter).—This paper should prove of considerable interest. The securing of an efficiency as high as 88% in a turbine of such small dimensions and of such low power and specific speed, is unusual. In the last few years we have become accustomed to efficiencies of more than 90% at the Holyoke testing flume, and results in excess of this in turbines of high power and of large dimensions, after installation. The dimensions of a turbine, as well as its power output, have a considerable bearing on the maximum efficiency which may be secured. For turbines of homologous design, the unit having the largest dimensions will develop the highest efficiency.

In the writer's experience, the Cornell turbine gives the highest efficiency ever secured in an authenticated test of a unit of such small dimensions and capacity. The testing of the turbine at different speeds, and the relations found between torque and discharge at various speeds, and at zero speed, are of considerable interest.

In the design of the Cornell turbine, the best proportions could not be used in every detail and, on account of utilizing existing patterns, the value of b was made abnormally large. This probably accounts for the part-gate efficiency not being as high as the excellent value of maximum efficiency would lead us to expect. In such small units, the design cannot be made geometrically similar in all respects to a large unit of the same specific speed. Fewer buckets must be used in the

* Hydr. Engr., I. P. Morris Co., Philadelphia, Pa.

runner in order to permit wider openings for the passage of trash; and, for mechanical reasons, a lower number of guide-vanes must be used, and the vanes are thicker in proportion. Therefore, the slight modifications in the design, which have been necessary because of the small size of the unit, would probably lower the part-gate efficiencies to a certain extent. Mr. Taylor.

The writer had an opportunity of examining the test data under discussion previous to the publication of the paper, and, with the exception of the head measurement, he is in entire agreement with the methods used. Some correspondence has passed between the author and the writer regarding head measurement, and, as there is still a disagreement over this point, the writer thought that in his discussion it would be advisable to confine his remarks entirely to this question, especially as there may be differences of opinion among engineers as to just what constitutes the correct effective head on a turbine. The author has very kindly given the writer permission to quote passages from his letters.

The method of measuring the head, as specified in the contract for this turbine, is as follows:

"In determining the efficiency of the unit, the head shall be measured in the following manner: four mercury gauges shall be attached to the penstock near the intake flange of the casing, the gauges being equally spaced around the circumference. The average reading of these gauges, reduced to feet of water, when added to the vertical distance between the height of the mercury in the pressure side of the gauges and the level of the tail-water at the discharge from the draft-tube, shall be considered the head, after being corrected for the difference in velocity at the point where the gauges are attached and at the discharge from the draft-tube."

The writer contends that this is the correct method of measuring the head, as it charges against the turbine all losses due to friction, eddies, whirls, etc., from the beginning of the turbine (the intake flange of the casing), to the end of the turbine (the end of the draft-tube). It does not, however, nor should it, charge against the turbine energy which would not be effective in the turbine, even should the efficiency of the latter equal 100%, such as the losses external to the turbine, or which occur outside of the limits specified.

There is no disagreement with the author over the question as to whether a turbine should be charged with the losses through the racks, intake, and penstock, for all agree that it is the efficiency of the turbine which is to be determined and not the over-all efficiency of the plant, which is a different matter. The disagreement arises solely from the question as to whether the turbine should be charged with the energy represented in the velocity at the exit from the draft-tube.

Mr. Taylor.

Three or four years ago this question was frequently discussed with engineers of power companies during the preparation of contracts. In the past three years, however, there has been no disagreement between the various power companies and the writer in regard to the measurement of the effective head on the turbine. In every contract it has been stated clearly that the velocity head at the exit from the draft-tube will not be charged against the turbine.

In Paragraph 5 of the paper the author states that the method of estimating the head in a turbine test is a matter of agreement in any case. This is perfectly true, but there is no reason that the method agreed on cannot represent the true effective head.

The following are quotations from one or two of the author's letters regarding this question:

"You will note that in my paper I computed the head as I did in my report of the test, and not as specified in the contract. Some correspondence passed between us on this subject and I have since taken the question up with several others and find my contention borne out by them.

"In my letters I stated the question thus: 'Is it more logical to use (6) [method outlined in paragraph 6] or to subtract from that the velocity head at exit from the draft-tube'.

"One reply received is as follows: (*a*) 'I have your letter and wish to say that the logical and customary way of measuring the head under which a turbine is acting is that mentioned under (6)'." [That is, paragraph (6), without this velocity head correction.]

The only case where the velocity head from the draft-tube into the tail-race could be deducted would be when making comparisons of two turbines in which, owing to local conditions, this discharge speed had to be made widely different.

"From the Holyoke Water Power Company I received: (*b*) 'We would say that practically all our tests are made in open flume, and that the head is always taken to be the actual distance between the water surfaces in the flume and in the tail-race. We have never made any deductions whatever from this observed head. From our view, the draft-tube is a necessary adjunct to the wheel, and the velocity head therein is directly chargeable to the wheel'.

"In addition, I received a verbal comment to the same effect from Professor R. C. Carpenter.

"You understand that I do not maintain that your position is entirely wrong, for I see the merits of the case where it comes to comparing two turbines under widely different settings. But why not eliminate the draft-tube altogether, as in paragraph (5) of my paper? I should like to have you bring up this point also [in the discussion]. My view is that you are proceeding only half-way when you wish to leave out the velocity head from the draft-tube."

Referring to this last quotation, namely, the elimination of the draft-tube entirely, it is possible to give the author's views on this

matter a little more clearly by quoting from a letter from him dated November 9th, 1914, as follows: Mr. Taylor.

"I admit that there is merit in your contention for the reason stated by you and the correspondent quoted [see paragraph (*a*) above]. That is to say, the same turbine set in the two different ways shown would give different efficiencies due to the superior draft-tube in the one to the left [referring here to Fig. 13].

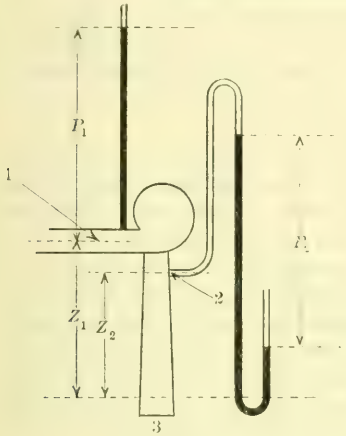

FIG. 13.

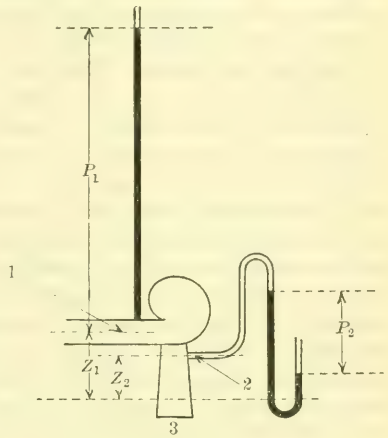

FIG. 14.

"Yet the difference is not in the turbines themselves, but in their draft-tubes. It is not a question even of draft-tube design, but is simply one of setting. But it seems to me that the only true way to compare turbines is to eliminate the draft-tube altogether. Thus we would say

$$h = z_1 - \frac{v_1^2}{2g} + p_1 - z_2 - \frac{v_2^2}{2g} - p_2.$$

"In that event, our efficiency would be the same for both. The difference between this and $h_1 = z_1 + \frac{v_1^2}{2g} + p_1$ is equal to the friction loss in the draft-tube plus the discharge loss, $\frac{v_3^2}{2g}$. If you subtract the velocity head only for each one of the above figures [Figs. 13 and 14], you do not subtract as much for the left one as for the right-hand one. (Yet, due to the greater length and, possibly, greater leakage due to greater suction, the losses in the left suction tube (2 to 3) are greater than the right-hand one.) That would make the turbine on the left appear to give a slightly lower efficiency than the one on the right, unless the draft-tube losses were deducted also. The subtraction of $\frac{v_3^2}{2g}$ from the head at (1) is only a partial step toward the true result needed for comparison."

Mr. Taylor.

In reference to the quotation (*a*) given above, the writer disagrees with the author of the statement. The charging of the exit velocity from the draft-tube against the efficiency of the turbine is neither logical nor customary. It may be the method used by the party quoted, but the reverse has been the case in the testing of the turbines in a very large majority of the more prominent installations in the United States and Canada during the past three or four years.

Nor does it appear to be a logical procedure, for the energy represented by the velocity at the outflow from the draft-tube cannot be utilized by the turbine. Therefore, why should it be charged against the turbine, any more than the energy represented by the losses through the racks or in the penstock?

It may be argued that it is within the province of the turbine builders to control the magnitude of this loss, and that it can be kept within negligible values by a proper design of the draft-tube; also, as the turbine builders are responsible for the design of the draft-tube, this loss should be charged against the turbine. In answer to this, it may be stated that, in the writer's experience, he has often been restricted in the dimensions of the outflow sections of the draft-tube, due very often to the desire of the power companies to keep the dimensions of the power-house as small as possible, and also to avoid rock excavation. It cannot be said that the discharge area of the draft-tube can be fixed by the turbine builder in every case.

It must be borne in mind that in first-class installations the velocity head at the discharge from the draft-tube does not exceed from one-quarter to three-quarters of 1% of the total head acting on the turbine, so that it would seem on first thought that the question is not a serious one. The writer, however, will endeavor to point out that there are serious aspects to this question of head measurement which deserve the careful consideration of hydraulic engineers.

Referring to the quotation (*b*), given above, from the Holyoke Water Power Company: This does not answer the author's question. Because the Holyoke Water Power Company has used, for a great number of years, a certain method of head measurement, it does not follow that it is the correct one, and it cannot be said that the methods used at the Holyoke flume are by any means scientific.

Referring again to the same quotation, the statement that "the draft-tube is a necessary adjunct to the wheel, and the velocity head therein is directly chargeable to the wheel" is misleading, and does not answer the author's question.

The draft-tube is not only a necessary adjunct of the wheel, but is a vitally important one. This fact is not disputed. The question is not whether the velocity head "within" the draft-tube should be directly charged to the wheel, but whether the velocity head at the exit from the draft-tube should be chargeable to the wheel. There is a

great difference between the velocity head at the top of the draft-tube, which in low-head installations may represent 50% of the total head acting on the turbine, and the velocity head at the exit from the draft-tube, which may represent only one-quarter of 1% of the total head acting on the turbine. Mr. Taylor.

In the extracts from the author's letters it is claimed that it is impossible to compare properly the efficiency of two turbines unless the draft-tube is eliminated entirely, and, therefore, the subtraction of $\frac{v_3^2}{2g}$ from the value of h_1 in the equation,

$$h_1 = z_1 + \frac{v_1^2}{2g} + p_1,$$

as suggested by the writer, is only a partial step toward the true result needed for comparison.

To illustrate this point, the author has used for comparison two turbines, one of which is located closer to the tail-water than the other, and he points out that in making a comparison between these two, a discrepancy will exist, due to losses in the draft-tube of the turbine located at the greater distance above tail-water, which are not present in the draft-tube of the turbine located nearer to the tail-water, resulting in a higher efficiency for the latter than for the former. These losses are due to air leakage, caused by greater vacuum, and also other losses, including friction, due to the greater length of tube.

It is difficult to discuss these suggestions of the author with any degree of satisfaction, before coming to some agreement as to the relation of the draft-tube to the turbine. As already pointed out, the writer contends that the draft-tube is a part of the turbine, as much as the casing, the guide-vanes, or the runner. Any losses due to friction, eddies, whirls, improper rate of change of section in the casing, guide-vanes, or runner, are certainly charged against the turbine. It is the function of the casing to convey efficiently the water from the penstock to the guide-vanes, with as little loss of energy as possible. It is the function of the guide-vanes to transmit this energy as efficiently as possible to the runner, and of the runner to convert this energy into mechanical energy on the shaft. It is the function of the draft-tube to regain the energy at the outflow from the runner as efficiently as possible. If the draft-tube is incorrectly designed, there will be losses within the tube in the form of eddies, whirls, caused by improper change of section, etc., just as there may be losses from these same causes in the casing, guide-vanes, or runner. The writer fails to see just why the draft-tube should be considered in a different light from the casing, guide-vanes, or runner, or why there should be any doubt as to all losses absorbed by the draft-tube being considered in the same light as similar losses in the casing, guide-vanes, or runner.

Mr. Taylor.

The turbine builder should be responsible for the design of the draft-tube, and the efficiency which he is enabled to secure depends largely on the type which he uses. If the draft-tube is properly designed and there are, consequently, few losses in this part of the turbine, then the turbine should be credited with a high efficiency, due to an absence of these losses. On the other hand, if the turbine builder, through ignorance, inexperience, or other cause, permits the installation of a draft-tube with improper rate of diffusion, etc., the turbine should be charged with the resulting losses. The outflow loss from the end of the draft-tube, however, as has been stated before, is an entirely different matter. The energy represented is not, nor can it be, effective in the turbine, and therefore it should not be charged against the turbine.

The value of the losses in the draft-tube and also of the vacuum are, in the author's opinion, evidently dependent on the vertical distance from the turbine to tail-water. This is not true. The vacuum and losses within the draft-tube and the length of the tube are not dependent solely on the height of the runner above tail-water, for the question of velocity heads is equally important in establishing the value of the vacuum and the length of the tube.

The author is evidently laboring under the impression that if a turbine is situated near tail-water, the draft-tube will be made only sufficiently long to provide a proper seal below the tail-water level; the draft-tube being, therefore, simply a device for connecting the turbine with the tail-race. It may be pointed out, however, that this use of the draft-tube may be its least important function.

Aside from the question of connecting the runner to the tail-race, the design of the draft-tube is controlled by the necessity of diffusing the velocity of flow from the runner vanes at such a rate as to regain the energy represented therein efficiently.

In high-head turbines, in order to avoid corrosion, and for mechanical reasons, runners of low-speed characteristics are used. In turbines of very low head, in order to secure the highest possible speed of generator, runners of the highest specific speed are used.

With an increase in specific speed, the velocity head at the discharge from the runner buckets approaches a higher percentage of the total head acting on the turbine. Consequently, in a low-head turbine, the value of v_2 may be higher than in a high-head turbine. Therefore, the maximum vacuum in a draft-tube in a low-head turbine may be equal to or very much greater than that in a high-head turbine.

In order that the sum of the static head (the distance from the runner to tail-water level) plus the velocity head from the runner buckets shall not exceed the barometric column, it is necessary in the case of very high specific-speed runners, even under low heads, to place them very close to tail-water level. As a matter of fact, with the

extreme values of specific speed used for runners for very low heads, in order that the vacuum within the tube shall not exceed the barometric column, it may be necessary to place the runner below tail-water level. In this case also, the draft-tube is just as important, and may be more important, than when the runner is placed considerably above low tail-water level, as the head represented by the velocity at the discharge from the buckets may be, for example, a very large percentage of the total head acting on the turbine, and, in order to secure a high efficiency in the turbine, it is necessary to provide a proper design of draft-tube, which tube would be placed entirely below low tail-water level. Mr. Taylor.

In view of the foregoing, it may be said that it is possible that a turbine with a runner placed very close to tail-water level may have within the draft-tube a higher vacuum than a turbine placed a considerable distance above tail-water; also, the draft-tube in the former case may be considerably longer than in the latter case, as the length of the draft-tube is governed by the diffusion of the velocity involved, as much as in connecting the runner to the tail-race level. Therefore, the writer does not agree with the author's statement that the tendency toward leakage or the losses within the draft-tube depend on the elevation of the wheel above tail-water.

It follows, therefore, that, as friction and possible leakage losses are not determined by the vertical distance of the runner above tail-water, there is no error introduced by measuring the draft-tube as the writer advocates; namely, by deducting from the vertical distance from the runner to tail-water the velocity head at the outflow from the draft-tube. Therefore, in making a comparative test on two turbines, in which the losses in the draft head are widely different, the efficiency obtained by measuring the draft-tubes, as just described, or as recommended by the writer, will result in not only a true comparison, but in true absolute results respecting the actual efficiencies of the turbines.

Consider, again, the effect of specific speed on the draft-tube: As already pointed out, it is possible that the draft-tube designed to suit a low specific-speed, high-head turbine may be the same as for a high specific-speed, low-head turbine, and that the velocities entering the top of the draft-tube would be the same in both cases. It follows, necessarily, that the percentage of loss due to friction, and other losses in the draft-tube, would be proportionally greater in the case of the low-head turbine, than in the high-head turbine. Hence, the friction loss in the draft-tube is an inherent characteristic of the turbine under consideration, and depends on its specific speed. In view of the fact that there is such an intimate relation between the draft-tube design and the runner, especially in high specific-speed turbines, it would not be logical to consider one as being part of the turbine without the other.

Mr. Taylor.

There are so many different ideas among engineers regarding the function of the draft-tube, and the actual performance of the turbine depends so much on the design of the draft-tube, that it is only proper and just that it should be considered a part of the turbine.

The writer will endeavor to point out that, for practical reasons, it is impossible to eliminate the draft-tube, as suggested by the author, even if it were desirable to do so. In order to eliminate the draft-tube, it is necessary to secure the reading, p_2, or the pressure at the top of the draft-tube. It may be pointed out, however, that static piezometer connections to a vacuum gauge on the draft-tube are always subject to erroneous readings. Water at the discharge from the turbine runner is at all gates in a state of whirl about the turbine axis, which whirl may be either in the forward or backward direction, or a combination of both. Whether the piezometer connection is obtained from the wall of the draft-tube, or from a tube inserted into the stream, it is practically impossible to obtain reliable indications of the static pressure or vacuum actually existing. If piezometer connections are made in the wall of the draft-tube, the centrifugal force resulting from the whirl of the water, in either a well-designed or poorly-designed tube, will always tend to raise the pressure at this point. Thus the apparent vacuum available for the turbine will be less than the vacuum actually existing, and the turbine will be apparently operating under a lower head than is actually the case.

It is safe to predict that, in general, piezometer readings on the side of a draft-tube will not read within 5% of the actual pressure within the tube. This, therefore, illustrates the discrepancy between the values of the head, as given on page 2481,* and accounts for the low reading of 137.9 ft., the true value being reduced considerably by the false reading of the piezometer. In view of the fact that false readings result from this whirl, and that the amount of whirl may be entirely different in any two turbines, it follows that the method of eliminating the draft-tube, as suggested by the author, would not give even an approximately fair comparison.

Consider, also, another case, where the method of comparing turbines by the elimination of the draft head would give erroneous results; namely, when the turbine is at too great a height above tail-water. This effect can be shown more clearly by referring to Figs. 13 and 14, which have been reproduced from the author's letter of November 9th, 1914.

Let p_{at} = the height of the barometric column of water;

Let p_{vt} = the vapor tension of the water; in other words, the absolute pressure at which water, at the temperature of that passing through the wheel, will boil or vaporize;

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Taylor.

Let L = the friction, eddy losses, etc., in the draft-tube. Then the greatest possible value of p_2, is

$$p_2 = p_{at} - p_{vt}$$

In this case p_2 is the actual pressure within the draft-tube, and not the apparent pressure as determined by a piezometer. Therefore, if the turbine is placed above tail-water so that

$$z_2 + \frac{v_2^2}{2g} - \frac{v_3^2}{2g} - L$$

is greater than

$$p_{at} - p_{vt}$$

the effective head on the turbine will be reduced by the difference. The turbine builder usually decides on the elevation of the unit above tail-water and, therefore, should be charged with this loss of head. The method of measuring the draft head by a piezometer would not in this case indicate the existence of this excessive draft head, and the turbine builder would not be charged with this head which is wasted; though, as a matter of fact, he should be charged with this loss. This same remark, of course, would apply to a comparison of turbines. Should the draft head of one of these turbines be just up to the limit, and that of the other excessive, the resulting comparison, of course, would be correspondingly erroneous.

It is somewhat surprising to find that a great number of turbines, installed previous to three or four years ago, have excessive draft heads. Even in some of the more modern plants, this condition is found, and, if the head should be measured in the manner proposed by the author, this loss in head due to the excess draft head would not be discovered, and the turbine would get credit for very high efficiency when, as a matter of fact, the high efficiency would not exist.

The writer finds* that the draft head on the 17 000-h. p. turbines at the Tallulah Falls plant of the Georgia Railway and Power Company (head 580 ft.), at full load is calculated to be at least 39.5 ft. The difference between this value and the barometric column, for the particular elevation at which this plant is located, which it is known could not exceed 34 ft., would mean a loss of at least 5½ ft., or practically 1% of the total head on the turbine.

It is very evident that the author's suggested method for the elimination of the draft-tube could not be applied to this case without showing an efficiency in the turbine which does not exist.

In the foregoing the writer has endeavored to point out that it is correct and proper to credit the turbine with the energy represented by the velocity from the end of the draft-tube; and, to meet the possible objection that the turbine builder might restrict the discharge diameter

* From information in the *Engineering Record*, March 28th, 1914.

Mr. Taylor. of the draft-tube purposely and unnecessarily, it would be easy to insert a clause in contracts to the effect that the discharge velocity head is not to amount to more than a certain percentage of the head, say one-quarter or three-quarters of 1%, as the case may require. The efficiency of the Cornell turbine is higher by 0.2% than the maximum given by the author, for the turbine has been charged with the energy represented by the exit velocity from the draft-tube, which energy it did not absorb.

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PAPERS AND DISCUSSIONS

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PENSTOCK AND SURGE-TANK PROBLEMS.
Discussion.*

By Messrs. Irving P. Church, R. D. Johnson, John C. Trautwine, Jr., and Gardner S. Williams.

Mr. Church.

Irving P. Church,† Assoc. Am. Soc. C. E. (by letter).—This paper contains a valuable résumé of formulas useful to hydraulic engineers. Among those for the maximum surge in a surge tank, as due to a sudden full opening of the turbine or penstock gates, Equation (*G*) is referred to as "the most nearly accurate," this designation being adopted, it may be supposed, on account of the exact mathematical steps taken in its derivation by Dr. Webster; but, as this derivation‡ involves an assumption which is rather far from the truth, namely, that the friction head is proportional to the velocity, instead of the velocity squared, the propriety of such a designation may perhaps be questioned. It may be of interest, therefore, to compare results obtained from this formula with those based on the latter and more accurate relation between friction head and velocity.

The writer, therefore, has undertaken the solution of the numerical example treated on page 2552,§ using the same method as in his discussion|| of a paper by Mr. R. D. Johnson. This method involves plotting a curve from its differential equation and making use of the

* This discussion (of the paper by Minton M. Warren, Jun. Am. Soc. C. E., published in October, 1914, *Proceedings* and presented at the meeting of December 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Ithaca, N. Y.

‡ Dr. Webster's derivation of Equation (*G*), as given in the paper, seems to have followed the same procedure as that pursued in Professor Prasil's article in the *Schweizerische Bauzeitung* of 1908 (Nov., Dec., pp. 271, 301, 317, and 333). In this article the assumption is made that friction head is proportional to the velocity in order to render integrable the differential equation involved. (Professor Prasil's article, as translated for *Canadian Engineering*, and printed in that journal during August and September, 1914, may now be procured in pamphlet form.)

§ *Proceedings*, Am. Soc. C. E., for October, 1914.

|| *Journal*, Am. Soc. of Mech. Engrs., June and October, 1908.

Mr. Church.

relation that friction head is proportional to the velocity squared. Some of the details and results will now be presented, with further deductions. The author's notation will be retained, with foot-pound-second units in the numerical work.

The data of this example will be taken as: $Q = 112$, $P = 19.6$, $L = 13\,000$, $A = 154$, $F = 20.5$, $V = 5.7$, and $Y = 440$, there being one slight deviation from the author's data, namely, 20.5 for F, instead of 20. The reason for this is that at first the writer took the phrase "head lost" (in the author's definition of F) too literally, as meaning the loss of head due to pipe friction plus that due to entrance resistance, on which basis the vertical distance of the water surface in the surge tank from that of the forebay during the final steady flow would be 20 ft. $+ \frac{V^2}{2g} = 20.5$ ft., which quantity entered into the writer's formulas. Later, a careful reading of other parts of the paper showed that the author intended his F to include the $\frac{V^2}{2g}$; but by that time so much tedious work had been done that the writer retained the value of 20.5 for the author's F and, accordingly, solved Equation (G) with $F = 20.5$, to get Dr. Webster's result for S.

If we write Dr. Webster's Equation (26) in the form

$$A \frac{d\,s}{d\,t} = P\,(V - v)$$

and eliminate $d\,t$ by the aid of Equation (24), there is obtained

$$s\,d\,s - \frac{F}{V^2} v^2\,d\,s = \frac{L}{A}\frac{P}{g}[V\,d\,v - v\,d\,v] \ldots\ldots\ldots (100)$$

in which there are only two variables, s and v. Integrating each term, between the limits, 0 and S, for s, and the corresponding limits, 0 and V, for v (since, when $s = S$, the rate of flow, Pv, in the pipe is equal to Q, that is, v is then equal to V), we have

$$\frac{S^2}{2} - \frac{F}{V^2}\int_0^S v^2\,d\,s = \frac{L}{A}\frac{P}{g}\left[V^2 - \frac{1}{2}\,V^2\right] \ldots\ldots\ldots (101)$$

that is,

$$S^2 - \frac{2\,F}{V^2}\int_0^S v^2\,d\,s = \frac{L}{A}\frac{P}{g}\,V^2 \ldots\ldots\ldots\ldots (102)$$

Equation (102) embodies the relation that friction head is proportional to the velocity squared, which it is intended to retain in order to compare the results with Equation (G).

The integration, $\int_0^S v^2\,d\,s$, is only indicated, and could be rigorously carried out only by knowing v as a function of s, which function is as yet unknown.

Mr. Church.

In Fig. 3 let $O\ H\ K$ be the vertical axis of the surge tank, O being the position of the water surface in it (static level) at the beginning of the surge due to a sudden full opening of the turbine gates, and H the unknown position of this surface at the end of the first downward surge, that is, $\overline{OH} = S$.

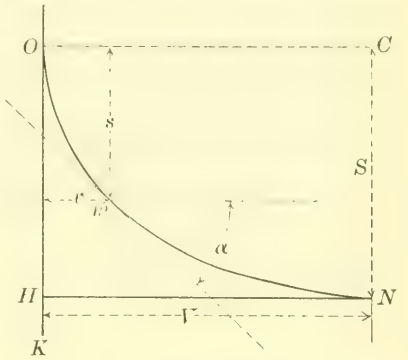

Fig. 3.

At any instant during the surge the water surface is some distance, s, below O, and the water in the main pipe has some velocity, v. If v be plotted as the horizontal co-ordinate and s as the vertical, with origin at O, the curve, $O\ m\ N$, is the result. In any numerical case this curve can be plotted quite accurately from its differential equation, which we already have in Equation (100), and which may be thrown into the form,

$$\frac{d\,s}{d\,v} = \frac{L\,P}{A\,g}\,\frac{V - v}{s - \dfrac{F}{V^2}\,v^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (103)$$

This gives the value of the "slope", or tan. α, for any point, m, of the curve (with due regard to scales).

Now, at the point, O (beginning of surge), both s and v are zero, so that tan. α = infinity, and hence the tangent line at O is vertical; and, at the end of the surge (point N), $v = \dfrac{Q}{P} = V$, and tan. α is zero, showing that the tangent line at N is horizontal (note that these positions of the terminal tangents are also found in the quadrant of an ellipse). Starting at O, then, we may plot the curve by its differential equation (successive tangents, with points of tangency). This was done by the writer, using co-ordinate paper of $\frac{1}{20}$-in. "mesh". On the drawing thus made, $\overline{OH}$ and $\overline{ON}$ were lengths of some 21 and 28 in., respectively.

By this curve a value of 43.25 ft. was found for S. From the curve there was also found, by numerical integration,

$$\int_0^S v^2\,d\,s = 0.1138\ V^2\,S \qquad \ldots\ldots\ldots\ldots\ldots\ldots (104)$$

which, substituted in Equation (102), gives the quadratic,

$$S^2 - 0.228\ F\,S = \frac{L\,P\,V^2}{A\,g} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (105)$$

Mr. Church.

the solution of which gives the same value, 43.25, for S, which is a good check on the plotting of the curve. For the same numerical data, Dr. Webster's Equation (G) gives $S = 46.08$ ft.; and, if friction is neglected (that is, with $F =$ zero), $S = 40.87$ ft. It would appear, therefore, that Equation (G) gives an increase in the surge as due to friction about double that based on the relation of the velocity squared.

Regarding Equation (105) (based on the plotted curve) as fairly general, we find, solving for S,

$$S = 0.114\ F + \sqrt{\frac{L\ P\ V^2}{A\ g} + 0.0130\ F^2} \quad \ldots\ldots\ldots (106)$$

which, as the second term under the radical is generally quite small compared with the first, may be written

$$S = \frac{1}{9}\ F + \sqrt{\frac{L\ P\ V^2}{A\ g}} \quad \ldots\ldots\ldots\ldots\ldots (107)$$

and this would probably give a fair approximation in most practical cases. With the numerical data just treated we find $S = 43.19$ ft., from Equation (107) [as compared with 43.25 ft., from Equation (106)].

Noting that a quarter ellipse, plotted to S and V as semi-axes, deviated at no point more than slightly from the true curve, the writer thought the former curve might be a fair substitute for the latter in all cases, so far as obtaining the value of the "fractional" integral, $\int_0^S v^2\ d\ s$, in terms of V^2 and S is concerned, which being done, Equation (102) becomes a quadratic easily solved for S.

The equation of the quarter ellipse being

$$S\ v = V\left[S - \sqrt{S^2 - s^2}\right] \quad \ldots\ldots\ldots\ldots (108)$$

the summation, $\int_0^S v^2\ d\ s$, was found to be $\left(\frac{5}{3} - \frac{\pi}{2}\right) V^2 S$, and this, in Equation (102), gave

$$S^2 - 0.1917\ F\ S = \frac{L\ P\ V^2}{A\ g} \quad \ldots\ldots\ldots\ldots (109)$$

and hence, finally,

$$S = 0.0958\ F + \sqrt{\frac{L\ P\ V^2}{A\ g} + 0.00918\ F^2} \quad \ldots\ldots\ldots (110)$$

From this equation, with data of the numerical example just treated, we derive $S = 42.70$ ft.

Again, without notable error, this "elliptical" formula might be replaced by

$$S = \frac{1}{10}\ F + \sqrt{\frac{L\ P\ V^2}{A\ g}} \quad \ldots\ldots\ldots\ldots\ldots (110)$$

Mr. Church.

It may be of interest to apply Equation (106) to the data of some experiments performed in 1910 by Mr. H. H. Conway and F. B. Storey, Jun. Am. Soc. C. E., in their thesis work at the Hydraulic Laboratory* of the College of Civil Engineering, Cornell University, and compare the results with those of the experiment.

A pipe, 140 ft. long and 4 in. in diameter, leading out of a pond, was furnished at the lower end with a vertical stand-pipe, and also a valve, opening into the atmosphere; and the effect of the more or less sudden or complete opening, or shutting, of the valve on the water column in the stand-pipe (which was hardly wide enough to be called a surge tank, perhaps, but was intended to represent the latter) was noted.

In one experiment, involving a sudden full opening of the valve, the water in pipe and stand-pipe being originally in a static condition, the stand-pipe was 6 in. in diameter, and the pipe 4 in. With steady flow, and valve fully open, the velocity in the pipe would have been $V = 7.5$ ft. per sec.; also, $F = 10.6$ ft. With these data and a length of 140 ft., we obtain, from Equation (106), $S = 11.7$ ft. The experimental result was 11.6 ft.

Another experiment was made involving a sudden full opening of the valve, with the same data as before, except that the stand-pipe was 4 in. in diameter (that is, no wider than the main pipe), with $F = 10.8$ ft. and $V = 7.5$ ft. per sec. In this case, Equation (106) gives 16.93 ft. for S, as compared with 14.4 ft. by experiment. This rather considerable discrepancy may perhaps be explained by supposing that, on account of the small size of the stand-pipe, the assumption that it could deliver water at a constant rate to the valve during the whole time of surge was rather wide of the truth.

As another matter of interest in connection with Mr. Warren's paper, an experiment† in "ordinary water-hammer", made by Mr. B. F. Latting in 1894 at Cornell University, may be referred to. The pipe used was 8 in. in diameter, 2 395 ft. long, and was furnished at the discharging end with a nozzle 2 in. in diameter, provided with a valve gate, the static head being $Y = 302$ ft. There were two experiments, intended to be alike in conditions, in which the valve gate was gradually closed (after steady flow had once been started) and in such a way (so intended) as to produce a uniform negative acceleration (that is, retardation) in the velocity, v, in the pipe. The proper motion of the gate to produce this effect was investigated theoretically (see Equation (112)), but only gravity, friction, and the inertia of the water were brought into play, no account being taken

* In charge of E. W. Schoder, Assoc. M. Am. Soc. C. E.

† Described with related matters in the *Transactions*, Assoc. of Civ. Engrs. of Cornell University, for 1898, p. 31. The same article, "Unsteady Flow of Water in Pipes," was reprinted in the *Cornell Civil Engineer* of December, 1908, p. 61.

Mr. Church.

of its elasticity, because the time of closing, $T = 25$ sec., was much greater than $\frac{2L}{a}$. The corresponding rate of decrease of gate area was not linear, with respect to the time, as Allievi's theory requires, but nearly so, with the important exception, however, that at the beginning of the closing an instantaneous movement of the gate was demanded through a small portion of its whole range. This, of course, was difficult to approximate to in the experimental handling of the gate. The initial velocity of the water in the pipe was $V = 6.4$ ft. per sec. The pressure at the lower end of the pipe (near the base of the nozzle) was observed at regular intervals.

Neglecting the elasticity of the water and the pipe, we have, at any instant of the closing, for the pressure (head) (above atmosphere) at the lower end of pipe,

$$y = Y - \frac{L}{g}\frac{dv}{dt} - \frac{F}{V^2}v^2 \ldots\ldots\ldots\ldots\ldots\ldots (111)$$

We have, also, at any instant, the relation :

$$y = \frac{v^2}{2g}\left[\left(\frac{P}{A'}\right)^2 + \zeta\left(\frac{P}{A_0'}\right)^2 - 1\right] \ldots\ldots\ldots\ldots (112)$$

in which A' = the variable gate area (opening), A_0' being its value at the beginning of the closing, while ζ is the coefficient of resistance of the nozzle (0.37). If an assumption is made for $\frac{dv}{dt}$ as a function of t, Equation (112) can be used to determine what function A' must be of t to produce the assumed relation for $\frac{dv}{dt}$.

If the retardation of the water in the pipe is to be uniform, we have $\frac{dv}{dt} = -\frac{V}{T}$, whence

$$y - Y = +\frac{L}{g}\frac{V}{T} - \frac{F}{V^2}v^2 \ldots\ldots\ldots\ldots\ldots (112)$$

and hence, at the end of the closing, for the maximum $y - Y$, or h, we derive

$$h = \frac{L}{g}\frac{V}{T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (113)$$

For the data of Mr. Latting's experiment, this gives $h = 19.05$ ft., and Equation (B) of Mr. Warren's paper (with $\alpha = 4\,100$ ft. per sec.) gives 19.50 ft.; whereas, from Allievi's formula, Equation (C), we derive $h = 19.6$ ft. The experiment itself gave $h = 28.5$ ft.; but the gate was necessarily tardy in reaching the positions called for by theory near the beginning of the experiment, so that the retardation

of the water was less at the beginning than intended by theory, and hence somewhat greater near the close. Mr. Church.

Further details of the theoretical variation of pressure, velocity, area of gate opening, etc., during the closing, corresponding to various assumptions of the law of retardation of the velocity in a pipe, as a function of the time, may be found in the *Transactions* of the Association of Civil Engineers of Cornell University, already referred to. Numerical instances were also worked out, but the time, T, was so much greater than $\frac{2L}{a}$ that the compressibility of the water and distension of the pipe were not supposed to have any important effect on the results.

R. D. JOHNSON,* Esq.—The speaker was much interested in one point brought out by Mr. Warren, namely, that the dynamic pressure does not continue to rise throughout the entire operation of closing a gate. This statement on his part has led the speaker to refer to certain corroboratory testimony. Mr. Johnson.

For slow-closing gates there would seem to be no important error introduced if the mass of water were treated as a rigid or non-elastic substance, especially if, in so doing, a mathematically exact statement could be written for the effect of the discharging water during the closure. An arithmetic solution (Table 4) is offered for the case where $L = 402$, $Y = 100$, $V = 6$, and $T = 2.5$, corresponding to Mr. Warren's assumptions, except as to the value of T.

TABLE 4.

Time (t).	Gate (G).	Velocity (v_0).	Total Head (y_0).
0	0.60	6.00	100
0.25	0.54	5.74	113
0.50	0.48	5.30	122
0.75	0.42	4.75	127.5
1.00	0.36	4.13	131
1.25	0.30	3.46	133.5
1.50	0.24	2.78	134.0
1.75	0.18	2.09	134.5
2.00	0.12	1.39	134.5
2.25	0.06	0.69	134.5
2.50	0.00	0.00	134.5

The values in Table 4 are intended merely for illustration, and are not particularly accurate. The length was chosen as 402, in order that the retardation for 0.25 sec. would be an even 0.02 of the dynamic back pressure, h. The time intervals may always be chosen in such a way as to make the retardation discernible by inspection of the instantaneous value of h, and this is essential to facilitate the balancing process of the arithmetical work. The first figure, 0.60, under the heading, "Gate", is a number representing full gate opening, and

* New York City.

Mr. Johnson.

is chosen so that when it is multiplied by the square root of the total pressure head back of the gate, the product will equal the existing penstock velocity, thus, $G\sqrt{Y} = V$. Assuming uniform closure, the corresponding values of G are first written down opposite the time intervals. It is perhaps as well not to choose the time intervals shorter than $\frac{L}{a}$, because the whole mass cannot be affected within this time at any rate. A small error is introduced at the start, by this process, in assuming that the retardation for the first Δt is proportional to the dynamic pressure existing at the end of 0.25 sec., but this is smoothed out pretty well by subsequent computations. Of course, if $\Delta t = d t$, no error of this character would exist. It will be noticed by inspection that at any time $\Delta v_0 = 0.02 h$, and also, $G\sqrt{y_0} = v_0$, where G is for the time being regarded as a variable. This is the balance which must be effected by trial, but it is a very simple process if the relation between Δv_0 and h is made so straightforward as not to require any mental effort to convert them. Table 4 was worked out in a very few minutes with the aid of a 6-in. slide-rule. The speaker has worked out a large number of such computations in order to ascertain the power delivered to a water-wheel during gate motion, open or shut, and thereby determine the speed variation when working on a given fly-wheel effect. The total head may also be varied, to account for any variation in the stand-pipe, and friction may also easily be included if desired.

The speaker has not followed closely the literature on the subject of water-hammer, and almost fears to offer a formula which may be a stale one, but as it is quite easy to express exactly the foregoing process by the use of the calculus, it is thought that it may be of some interest to do so. Before proceeding with this, it is desired to call attention to the shape of the curve between t and h. The reader is asked to plot this from Table 4, and to notice how the pressure rises on a smooth curve, reaching a constant value in about 1.5 sec. If there is any virtue in this method, it shows that the time to reach the constant value has no particular relation to the critical time, $\frac{2L}{a}$. The speaker's impression is that the duration of the flat or constant part of this curve is a function of T, other things remaining unchanged, and that the larger the value of T, the sooner, proportionately, the flat is reached; and, therefore, the larger the proportion of T during which h remains constant. In Mr. Warren's numerical example, where $T = 5$, the time to reach $h_{max.} = 16$ is thus found to be 1.5 sec., which is also the time to reach $h_{max.} = 5$ when $T = 15$. Perhaps the actual critical time is independent of T, it looks as though this were so in this case, at any rate.

Development of Formula.—By hypothesis, as in the numerical integration, and using Mr. Warren's nomenclature, except as noted, Mr. Johnson.

$$V\sqrt{\frac{y_0}{Y}}\left(1-\frac{t}{T}\right)=v_0 \quad \ldots\ldots(1)$$

and

$$\frac{g}{L}h\,dt=-dv_0 \quad \ldots\ldots(2)$$

Differentiating Equation (1) and dropping the subscripts:

$$\frac{V\,dy}{2\sqrt{Y\,y}}\left(1-\frac{t}{T}\right)-\frac{V\,dt}{T}\sqrt{\frac{y}{Y}}=dv=-\frac{g}{L}h\,dt \quad \ldots\ldots(3)$$

or,

$$\frac{dy}{dt}=\frac{\frac{2\,V\,y}{T}-\frac{2\,g\,h\,\sqrt{Y\,y}}{L}}{V\left(1-\frac{t}{T}\right)} \quad \ldots\ldots(4)$$

Equating this to zero to get $y_{max.}$, we have,

$$\frac{V\,y}{T}=\frac{g\,h}{L}\sqrt{Y\,y} \quad \ldots\ldots(5)$$

substituting $y = Y + h$, and solving for h, we have,

$$h_{max.}=\frac{L\,V}{2\,g^2\,Y\,T^2}\left\{L\,V+\sqrt{4\,g^2\,Y^2\,T^2+L^2\,V^2}\right\}$$

or, if $2\,g\,Y\,T$ be put $= N$,

and $L\,V$ be put $= M$,

$$h_{max.}=\frac{2\,M\,Y}{N^2}\left\{M+\sqrt{M^2+N^2}\right\} \quad \ldots\ldots(6)$$

It may be noted that Equation (6) is identical with Mr. Warren's Equation (*C*), as accredited to Mr. Allievi, although this was at first overlooked on account of the dissimilarity in nomenclature.

Integrating Equation (3), we have an expression of the following form:

$$\log.\,x=n\log.\frac{T}{T-t} \quad \ldots\ldots(7)$$

$$\text{or, } t=T\left\{\frac{\sqrt[n]{x}-1}{\sqrt[n]{x}}\right\} \quad \ldots\ldots(8)$$

That is, the ratio of t to T equals the nth root of x minus 1 divided by the nth root of x.

$$\text{Where } x=R\left\{\frac{H+\sqrt{y_o}}{J-\sqrt{y_o}}\right\},\ R \text{ being } \frac{J-\sqrt{Y}}{H+\sqrt{Y}}, \text{ and}$$

$$n=\sqrt{\left(\frac{2\,g\,T\,\sqrt{Y}}{L\,V}\right)^2+1},\ H=\frac{L\,V}{2\,g\,T\,\sqrt{Y}}(n-1),$$

$$J=\frac{L\,V}{2\,g\,T\,\sqrt{Y}}(n+1).$$

Mr. Johnson.

It may be found convenient to note that $h_{max.}$ can be expressed with this nomenclature as $\frac{2\,J\,H}{n-1}$.

It becomes quite simple to substitute random values of y_0 in Equation (8) and solve for the corresponding values of t. This is made particularly easy if such values of T, Y, L, and V are chosen that n becomes an integer.

Of course, the values of y_0 must be chosen between the limits of Y and $Y + h_{max.}$ in order to get rational values of t.

The interested reader is asked to plot such a curve, and to note how the area under the curve, bounded by the co-ordinate axes, is equal to $\frac{L\,V}{g} + Y\,T$, as it should be. The speaker has done this for two or three random cases and has found the correct relation of $\int_0^T h\,dt = \frac{L\,V}{g}$, thus checking Equation (8).

It will be seen that the speaker's previous statement, in regard to a constant value of $h = h_{max}$ for the latter portion of the closing time, is not theoretically correct, because an inspection of Equation (8) shows that the absolute maximum value cannot be attained before $t = T$. Practically, however, in a large number of cases, h becomes so nearly constant that the numerical integration, previously illustrated, fails to detect the slight upward inclination of the pressure-time curve, beyond a certain more or less critical point.

Taking Mr. Warren's example and cutting the time in two, that is, letting

$$T = 2.5, \text{ Equation (6) gives } h_{max.} = 34.6,$$
$$\text{Mr. Warren's Formula } (B) \text{ gives } h_{max.} = 31.8.$$

In order to try out the effect of varying Y, which Formula (B) neglects, let Y be reduced materially, say, from 100 to 25, other conditions remaining as before;

$$\text{Equation (6) gives } h_{max.} = 52.5$$
$$\text{Equation } (B) \text{ gives } h_{max.} = 31.8$$

The former value is 165% higher than the latter, and, without attempting to make any recommendations at this time, it seems, at any rate, that the discrepancy is worth investigating.

It is found that the formula for critical water-hammer agrees with Equation (6) when $T = \frac{L\,V}{1.5\,g\,Y}$, so that evidently this equation applies only when $T =$ or $> \frac{L\,V}{1.5\,g\,Y}$. If T is less than this value, the formula for instantaneous closing should possibly be used.

Mr. Trautwine.

JOHN C. TRAUTWINE, JR.,* ASSOC. AM. SOC. C. E.—The speaker has a personal interest in this paper on account of its mention of the Joukovsky water-hammer experiments and of Miss Olga Simin's paper on "Water-Hammer", which was a translation, greatly amplified, of Joukovsky's account (in Russian) of his experiments.

Joukovsky's experiments were made in Moscow in 1897-98, while Miss Simin's father, Mr. Nicholas P. Simin, was Chief Engineer of the Moscow water-works.

In 1904, Miss Simin made her translation, and this was elaborated and extended by Mr. Boris Simin, in the office of the speaker, who was thus brought into intimate relation with their studies. He was particularly interested in the phenomena of the water-hammer wave, as described by Joukovsky and much more fully discussed and illustrated by Mr. Boris Simin in the paper mentioned. As it seems that these phenomena are not generally known, it may be interesting to describe them more fully.

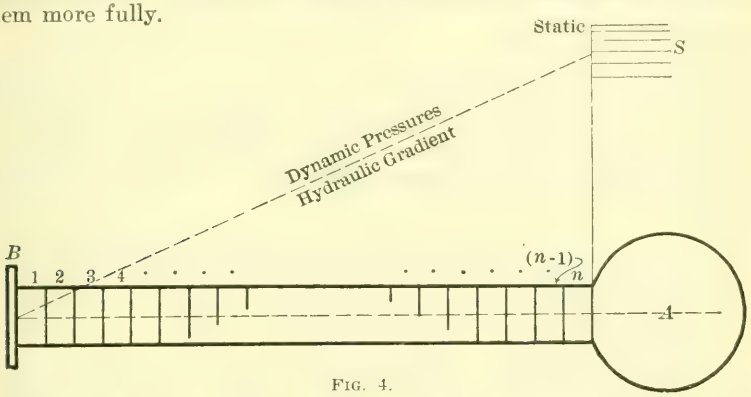

FIG. 4.

In Fig. 4, let the circle, A, represent a large reservoir, from which extends a long, straight, and horizontal pipe, $A\ B$, of uniform section; and let reservoir and pipe be full of water.

So long as the end, B, of the pipe remains open, the water flows from the reservoir, A, through the pipe, with velocity uniform as to both time and place, escaping freely at B. The pressure in the reservoir is always that due to the static head therein; but the pressures in the pipe (while the water escapes freely at B) decrease uniformly from a little less than static pressure, at a point, n, next the reservoir, to atmospheric pressure at the open pipe-end, B, as indicated by the hydraulic gradient. In other words, during flow, each cross-section of the pipe has its own proper dynamic pressure, depending on its distance from the origin, n. See the five diagrams at the top of Fig. 5; time = 0 sec.

* Philadelphia, Pa.

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Now let the gate at B be closed; and, for simplicity, let it first be supposed that the closure is instantaneous. Consider the water column, A B, as divided into n very thin and equal disks, 1, 2, 3, 4, . . . $(n - 1)$, and n.

The phenomena of water-hammer take place in a series of cycles, each consisting of four processes (and each process consisting of a conversion of kinetic into potential energy, or *vice versa*), as follows:

Process (1).—Disk 1, meeting, in the closed gate, an obstacle to its movement, is arrested and compressed. Its pressure is thereby raised above the static pressure, and the pipe-walls surrounding it are extended. The same takes place with each of the following disks, 2, 3, 4, . . . $(n - 1)$, and n, each, in turn, as it is stopped and compressed, acquiring the same superstatic or "high" pressure as Disk 1.

During Process (1), a small quantity of water flows from the reservoir, A, into the pipe, occupying the space which otherwise would be left by the compression of the water and the extension of the pipe-walls.

Finally, at the instant when all the disks (including the small additional quantity mentioned) have been arrested, all the water in the pipe is at uniform superstatic or "high" pressure, and at rest; and a wave of such pressure has traversed the length of the pipe, from gate to reservoir.

Fig. 5 traces the four processes of a first cycle in a pipe of such length that the pressure wave requires 4 sec. to traverse it in one direction from either end to the other. See the first and third, the third and fifth, etc., figures in the lower half of Fig. 5.

In Fig. 5, Process (1), just described, is illustrated by the three lower figures on the left; the figure at the extreme left (0 sec.) represents the instant after the gate is closed and just before the stoppage of flow begins; the second figure (2 sec.) represents the instant when the head of the first advancing wave of increased pressure has reached the middle point (2) of the pipe, the half of the pipe from (0) to (2) being then under superstatic or "high" pressure, while each point in the remainder of the pipe, and above the diagonal line, has still its original dynamic pressure, as indicated by the diagrams for points (3) and (4) (time = 2 sec.). The third figure (4 sec.) represents the end of Process (1), or the instant when the first pressure wave has traversed the entire length of the pipe from gate to reservoir.

The two solid arrows, next to the reservoir, between the first two figures, and between the second two figures, respectively, pointing from the reservoir, indicate the flow of water from reservoir to pipe during Process (1); the two dotted arrows near the middle of the pipe, with their plus signs, indicate the passage of the first pressure wave (a wave of increased pressure) through the pipe, from gate to reservoir.

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Process (2).—Referring again to Fig. 5: At the end of Process (1), the water next to the reservoir (Disk n, Fig. 4), being at higher than the static pressure in the reservoir, expands, and the pipe-wall surrounding it contracts, and the surplus water of Disk n, received

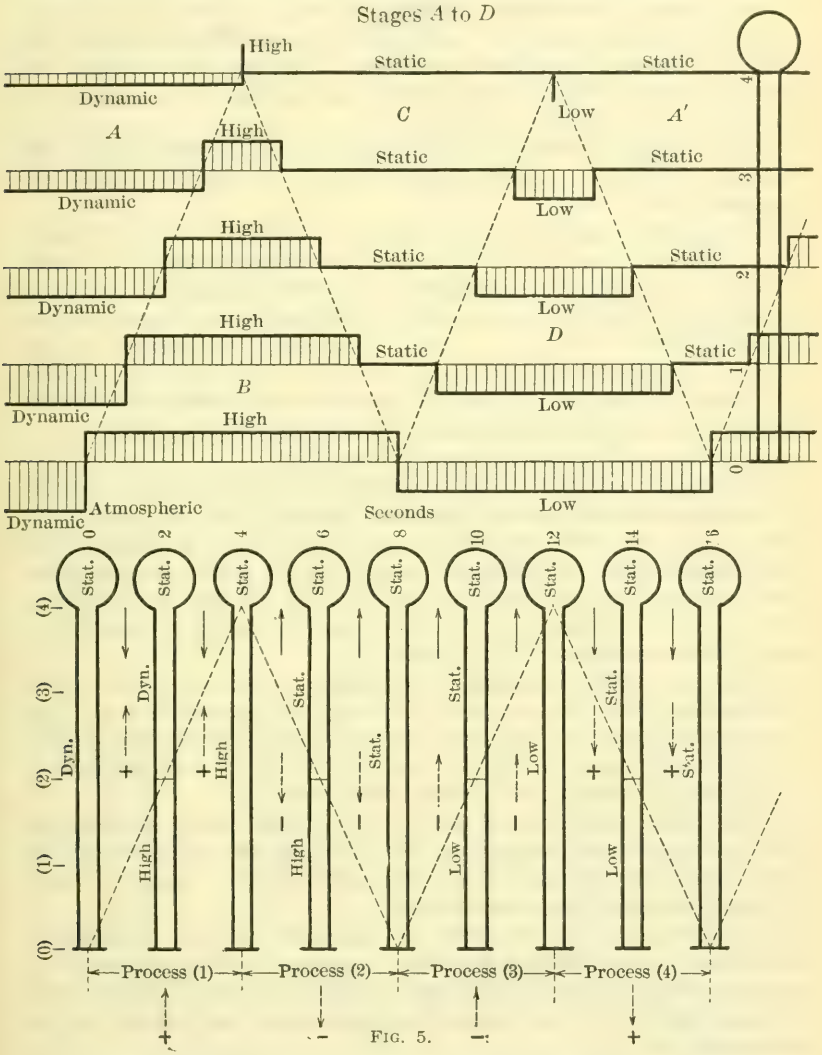

FIG. 5.

during Process (1), is forced back into the reservoir. The water of Disk n thus falls to static pressure; and this operation is repeated in each disk, $(n - 1)$, $(n - 2)$, etc., in turn, until (fifth figure, 8 sec.) it has affected Disk 1, at the gate. The wave of reduced (static)

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pressure has then traversed the length of the pipe, from reservoir to gate.

The two solid arrows, next to the reservoir, between the third and fourth figures (4 and 6 sec.), and between the fourth and fifth figures (6 and 8 sec.), respectively, indicate the flow of water back into the reservoir during Process (2); and the two dotted arrows, near the middle of the pipe, with their minus signs, indicate the passage of the wave of reduced (static) pressure through the pipe from reservoir to gate.

At the end of Process (2) (fifth figure, 8 sec.), all the water in the pipe is at uniform static pressure, but is in motion toward the reservoir; and

Process (3).—As there is no force to oppose this motion, it necessarily continues; and Disk 1, next to the gate, is left at a "low" pressure, theoretically as much below the static pressure as the static is below the "high" pressure; and, this taking place with each of the following disks, 2, 3, 4, . . . $(n - 1)$, and n, in turn, a wave of further reduction of pressure, to substatic, or "low", traverses the pipe from gate to reservoir. Note the solid flow and dotted wave arrows between the fifth and sixth figures (8 and 10 sec.) and between the sixth and seventh figures (10 and 12 sec.).

Process (4).—At the end of Process (3) (seventh figure, 12 sec.), the entire pipe sustains substatic or "low" pressure, and the water is at rest; but, the reservoir water being at a higher (static) pressure, flow again begins from the reservoir into the pipe, static pressure being thus re-established in Disks n, $(n - 1)$, . . . 4, 3, 2, 1, in turn, and a wave of increased pressure traversing the pipe from reservoir to gate. This is indicated by the arrows.

At the end of Process (4) (ninth figure, 16 sec.), the water is all flowing toward the gate, as in the figure on the extreme left; and, the velocity being the same as in that figure, the cycle of four processes is completed, although the pressures are now static and uniform, instead of dynamic and varying, as in the first figure.

The cycle is now repeated, and so on, until the system is brought to rest by the action of resistances; but, although the intensities of the pressures rapidly diminish, their durations remain constant through all repetitions until they cease entirely.

Summarizing: The direction and algebraic sense of the pressure wave and the direction of water flow, in each of the four processes of a cycle, are as given in the first four columns of Table 5, and as indicated by the dotted arrows and the signs (plus and minus) at the bottom of Fig. 5.

It will be noticed that when (Processes 2 and 3) the *pressure wave* is one of *diminution* (whatever its *direction*), the *water flow* is toward the *reservoir,* and *vice versa* in Processes 1 and 4.

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The last two columns of Table 5 give the character of the energy (kinetic or potential) and of the pressure (dynamic, static, high, or low) existing throughout the whole length of the pipe at the beginning and at the end of each of the processes of the first cycle.

TABLE 5.

Process.	Wave.		Water flows toward:	Energy.	Nature of pressure in pipe.
	Direction toward:	Sense.			
(1)	Reservoir	Plus	Gate	Kinetic	Dynamic.
(2)	Gate	Minus	Reservoir	Potential	High.
(3)	Reservoir	Minus	Reservoir	Kinetic	Static.
(4)	Gate	Plus	Gate	Potential	Low.
(1′)	Reservoir	Plus	Gate	Kinetic	Static.
				Potential	High.

In the upper half of Fig. 5, the five influence diagrams, marked 0, 1, 2, 3, and 4, indicate, respectively, the histories of the corresponding selected points in the pipe; this history, for each point, consisting of the four stages, *A*, *B*, *C*, and *D*. Each stage is covered by the triangle enclosed between two of the dotted diagonal lines.

Thus, except at the gate, Point 0 (Disk 1, Fig. 4) and at the reservoir, Point 4 (Disk *n*, Fig. 4), the pressure, after the closing of the gate,

in stage	*A*	dynamic	until the point is reached by the head of the	advancing	wave of	increased	pressure, in Process	(1)	The pressure then	rises to high	and (see next line),
	B	high		returning		reduced		(2)		drops to static	
	C	static		advancing		further reduced		(3)		drops to low	
remains	*D*	low		returning		increased		(4)		rises to static	
	A′	static		advancing		further increased		(1′)		rises to high	
	B′	high		returning		reduced		(2′)		drops to static	
	C″, etc.										

Thus (the italics referring to the *horizontal* lines in the tabulation above), the pressure, in Stage *A*, remains *dynamic* until the point is reached by the head of the *advancing* wave of *increased* pressure, in Process (*1*). The pressure then *rises to high*, and (see next line) in Stage *B*, remains *high*, until . . . , etc., etc.

At the gate, Point 0 (Disk 1, Fig. 4), Stages *C* and *A′* (static pressure) are of zero duration; while, at the reservoir, Point 4 (Disk *n*,

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Fig. 4), Stages *B* and *D* (high and low pressure, respectively) are of zero duration.

In the foregoing it is assumed that the gate is completely closed. When the closure is only partial, the water, of course, continues flowing in the pipe; and, after such partial closure, the pressures, instead of being uniform (as in the case of complete closure), vary in intensity from a maximum, next to the reservoir, to a minimum, at the gate; and, for the datum, or "normal" pressure, to which the increases and diminutions of pressure are to be referred, we have, not the static pressure, as in the case of complete closure, but, at each point, the dynamic pressure proper to it by reason of its distance from the reservoir.

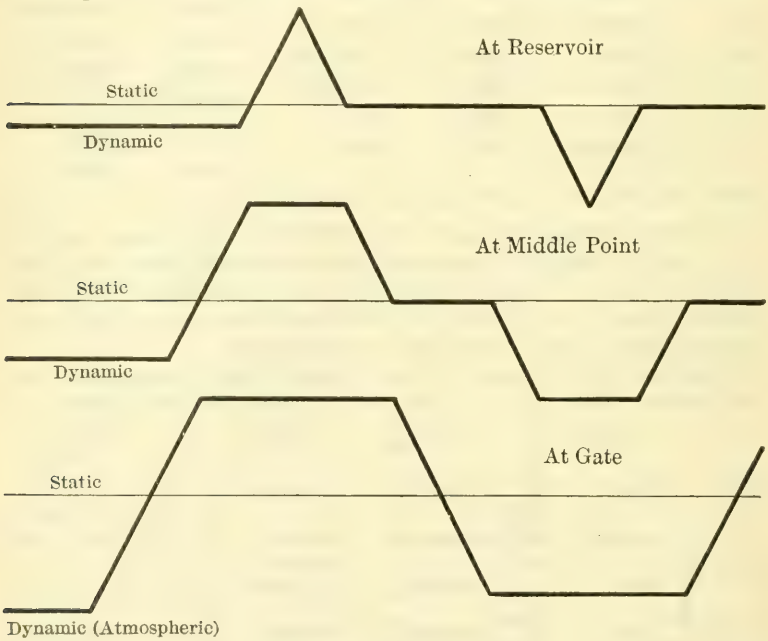

FIG. 6.

When (as in practice) the gate closure occupies a finite time, the horizontal lines of the diagrams, at the top of Fig. 5, are replaced by sloping lines, as in Fig. 6.

Analyzing the phenomena occurring in the case of such gradual closure, Mr. Boris Simin conceived of such closure as divided into a series of instantaneous partial closures, each having (for a given point in the pipe) its own separate influence diagram, analogous to those in Fig. 5, each such diagram having, of course, but a fraction of the total height, and each following its predecessor, and superimposed above it, the series forming a composite diagram, approximating to a straight

diagonal line, as the assumed partial closures are taken smaller and smaller. Mr. Trautwine.

It will be noticed that, in a given case, all points in the pipe are subjected to the same super-normal pressure, and to the same sub-normal pressure, independently of their distances from the reservoir; but that the several points sustain these pressures during periods of time ranging from zero (for the section at the reservoir) to the time required for one round trip of the pressure wave (for the section at the gate). This latter time is equal to that elapsing (for any point in the pipe) between an increase of pressure above the normal and the next decrease of pressure below the normal, or *vice versa*.

The autographic records of shock pressures, taken during the experiments, of course lacked the regularity of the ideal diagrams drawn for Fig. 5; but their character indicated the correctness of the foregoing statement of the phenomena of water-hammer.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E.—This problem is not solely a matter of pipes with which water-wheels are connected, nor is it one of the entire closing of gates. It is met frequently in the water distribution systems of manufacturing cities at about 5.30 or 6 P. M., the closing hour in large manufacturing establishments, when many of them, within a very few minutes, reduce their consumption of water; the resulting water-hammer in the pipes of the system becomes a very important matter, and, in several cases, has appeared to be the source of quite serious consequences. Mr. Williams.

It seems that the treatment of such a case is quite similar to that which has been presented by the author, except that it is apparently more complicated by reason of a somewhat irregular closing, and also by reason of the peculiarities of a distribution system, it being divided and connected in circulation. These conditions are likely to introduce elements that make mathematical treatment difficult, beyond an initial statement. Attention is called to the matter here, as water-works men have not always realized its full importance.

Papers of the nature of that submitted by the author, though difficult to understand and discuss, are really among the most valuable contributions to engineering knowledge. It is not particularly difficult to describe what has happened, but it is quite another matter to predict what will happen, and it is only by the aid of such studies as that of the author that engineers are qualified to attempt the latter.

* Ann Arbor, Mich.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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HISTORY OF LITTLE ROCK JUNCTION RAILWAY BRIDGE, OF THE ST. LOUIS, IRON MOUNTAIN AND SOUTHERN RAILWAY COMPANY, OVER THE ARKANSAS RIVER AT LITTLE ROCK, ARKANSAS, 1883-1914.

Discussion.*

By Messrs. Henry H. Quimby, M. L. Byers, Lee Highley, Theodore Belzner, and Robert H. Ford.

Mr. Quimby.

Henry H. Quimby,† M. Am. Soc. C. E.—It will probably be of interest to mention a timber crib pier foundation which is not solid, is more than 50 years old, is still doing its work satisfactorily, and probably will do so as long as it is wanted. The conclusions of the author are that timber cribbing is not suitable for bridge foundations unless it is solid.

Chestnut Street Bridge, over the Schuylkill River in Philadelphia, was built during the Civil War. The foundations of the pier in the middle of the river were laid in 1862. A crib of hewed square yellow pine, 29 ft. high, extended from the bed of the river to a point 2½ ft. below ordinary low water. The overlying gravel was dredged from the rock, and measurements were made. Then the crib was built at the side of the river to fit the contour of the rock, and towed to place and sunk with stone and gravel screenings in all the coffers. The masonry work was then commenced.

* This discussion (of the paper by C. E. Smith, M. Am. Soc. C. E., published in November, 1914, *Proceedings*, and presented at the meeting of December 16th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Philadelphia, Pa.

Mr. Quimby.

The reason that crib is doing its work satisfactorily is that it was built in a very different way from that used in the cribs described in the paper. The cross-timbers, spaced so as to leave coffers about 3 ft. square, were notched over each other at their intersections. Each stick was notched in one-quarter on each side, so that the timbers were theoretically in bearing throughout their length. The structure settled as the masonry superstructure of the pier was built on it, and this settlement was a little unequal, causing a slight list to one side. The total settlement is said to have been 6½ in. in the depth of 29 ft., and could be accounted for in part by the inaccuracy of the fit of the notches, and in part by the compression of the timber, the roughnesses of the hewed sides probably causing much of it.

A rough calculation, making guesses as to the total load that the crib is carrying, and the quantity of timber it contains, indicates that the compressive stress in the timber is less than 100 lb. per sq. in. The average pressure on the crib described in the paper, according to the author's calculation, and supposing the load to be central, is about 400 lb. per sq. in. of intersection bearing. The drawings show the timbers merely resting on each other at the intersections, and not notched over. The eccentricity of the load, as computed by the author, however, brings the maximum pressure up to 880 lb. per sq. in., which is entirely too much for side-grain bearing, and too much for end-grain for a continuing load, even on dry wood.

Wood appears to flow under pressure, the same as steel, and as concrete is found to flow. Therefore a long-continued pressure ought not to be as great as that of a transitory load. Bridge men have been brought up to regard live-load as more serious than dead-load stresses, but the reverse may really be the case, because a continued stress far below the rated ultimate strength may cause flow, and, as in the case of the Quebec Bridge, ultimate failure.

In a recent case of falsework, the vertical members were stressed, in some instances, up to 1 000 lb. per sq. in., and where the ends of the verticals bore on the side-grain of the sills and caps they crushed into it. The sills were blocked up, where adjustment was intended to be made, on blocks twice as long as they were wide, reducing the pressure on the blocks to one-half, or 500 lb. per sq. in., across the grain. The wood was saturated with water, the load was long continued, and quite a number of the blocks crushed. On some of them the sapwood split away from the heartwood, seemingly by reason of its swelling, under the action of the water; not being confined laterally, as it was vertically, it swelled in this direction as the blocks crushed.

The wood in the cribs described in the paper was evidently overloaded, and it simply crushed under the weight. The only wonder is that the engineers were not more alarmed about the condition of those piers. The illustrations show that it was very bad.

Mr. Quimby.

The most perplexing problems that structural engineers have to meet are generally found in such jobs as this—the repairing or strengthening of existing structures. The problem was met very successfully by the final strengthening.

Not the least interesting thing in this paper is the statement that the bridge superstructure, built 30 years ago, presumably for loads common in that day, is good for modern loads, and will be for many years to come. The probability is that that bridge, 30 years old, is of iron, not steel. Iron, in those days, was customarily stressed lower, in proportion to its ultimate strength, than steel is to-day, which accounts for the continued service of many old iron structures, including some of the elevated railways, under loads several times as heavy as were contemplated in the design.

Mr. Byers.

M. L. Byers,* M. Am. Soc. C. E. (by letter).—The object of this paper, as stated by the author, is praiseworthy, and in connection with the subject there are quite a number of points of interest to the Engineering Profession.

The writer was surprised, however, to find in the paper several of his signed official communications. As these communications are separated from their context, they are, perhaps, somewhat subject to misinterpretation. For example, the communication commencing on page 2718,* which is given in part, seems to require, for a complete understanding of the reasons governing its character, the knowledge that there was contained therein, among the various matters submitted, certain other tentative plans for general improvement in the vicinity which contemplated the entire abandonment of this bridge, and, before the final decision could be made as to the engineering features of the pier situation, it was necessary that these general development plans be finally passed upon by the management.

With reference to the effect of the temporary treatment of Pier 4 which was resorted to in 1908, the author seems to have changed his mind, as he states, on page 2710:†

"Following this work, no appreciable movement in the direction of the bridge was apparent for several months, but the movement at right angles to the bridge appeared to increase in rate, indicating that the work had no appreciable effect."

Whereas, in his memorandum to his superior, as of August 17th, 1909, which he has seen fit to quote on page 2713,† he states, "If the pier continues stationary, no action whatever is necessary, as the piers are all perfectly safe."

With reference to the yokes, the writer authorized their use with the express condition that they be constructed of such light section that it

* New York City.

† *Proceedings*, Am. Soc. C. E., for November, 1914.

Mr. Byers. would be impossible for them to exert sufficient additional stress on the bridge members to be of vital importance.

It is a source of considerable satisfaction to the writer that, due to the successful temporizing with this problem for a number of years, during and immediately following the 1907 panic, when money was scarce and saving imperative, and at the expense of a very considerable amount of worry to himself and staff, the opportunity has been afforded to the present management to solve this difficult problem successfully and economically.

Mr. Highley. Lee Highley,* Assoc. M. Am. Soc. C. E. (by letter).—Mr. Smith has given a very lucid and complete history of the trouble at the Little Rock Junction Bridge and its final correction. The writer obtained his first knowledge of the leading facts concerning this structure while working in a minor position in the Bridge and Building Department of the Missouri Pacific-Iron Mountain System. In fact, it sometimes fell to his lot to make the measurements for finding the movement of Pier 4. The earlier measurements were comparatively simple and devoid of elaboration. They took into account only the movement of Piers 3, 4, and 5 in the direction of the center line of the track and there was no determination of movement at right angles to this direction. From a fixed point in the center of the track behind the south abutment, called Pier 1, direct tape measurements were made along and over the ties to Piers 3, 4, and 5, using a plumb-bob from the top of the ties to chisel marks on the coping stones. A 15-lb. pull was put on the tape, and temperature allowance was made in calculating its length. It happened sometimes that the ties were wet from rain or possibly covered by a light fall of melting snow, and thus the temperature of practically one-half of the tape might differ considerably from that of the other half which was not in contact with the ties. In all cases where there was a suspicious difference between the results of any measurement as compared with previous ones the work was re-checked. The character of the weather, direction of the wind, and gauge reading of the river were also recorded. Despite the crude features, there was no getting away from the fact that Pier 4 was steadily moving in the direction it was leaning, through some periods slowly, through others with more acceleration.

Time went on, and conditions grew worse. In 1908, the observations included additional features. A system of cross transit lines, intersecting on the up-stream end of Pier 4, was used in addition to the straight tape measurement. Gas-pipe wood-filled hubs, with brass screw centers, were sunk flush with the surface of the earth about 600 ft. up- and down-stream from the Argenta, or north end, of the bridge. From these points a transit was sighted to fixed objects, corners of brick buildings, on the Little Rock side of the river. At the

* New Meadows, Idaho.

intersection of the transit lines a copper plug was fixed in the coping of the pier, and properly marked. From this intersection point lines were laid out at right angles and marked on the coping. One line was parallel to the center of the track and the other at an angle of 90° thereto. As the pier continued to move and future observations were taken, the point of intersection would necessarily fall within the quadrant determined by the two lines first laid down. By measuring the offset from the new intersection to the old lines, the movement of the top of the pier in two directions was determined. However, the transit observations, following this method, were not always satisfactory, were sometimes attended by difficult physical conditions, and did not always check reasonably close with the tape measurement. It should be remembered that the fore-sight objects were nearly half a mile away, across the river diagonally, and that fog and smoke often rendered it difficult to see. Also, as Mr. Smith points out, the continued movement of the pier soon brought bridge members across the line of sight and added to the complications. If the fixed time for observation fell on a dark, cloudy day it became necessary to use lights and reflectors to illuminate the plumb-bob cord, or small rod, which was used for obtaining points on the pier. The usual custom was to set three times on the fore-sight and give as many sets of points for determining lines, then take the mean of the three for the intersection line. Even under the most favorable light conditions, it was the writer's practice to take advantage of the sun's position, and lay one line of intersection in the morning and the other in the afternoon in order to get the truest reflection of light from the fixed fore-sight and the one on the pier. Mr. Highley.

The writer left the service of the Missouri Pacific System early in 1910, and has been out of touch with matters concerning the Little Rock Junction Bridge since that time. Therefore he finds this paper of unusual interest. The newer method of observing pier movements, as shown by Fig. 33, is much superior to those formerly used.

The plan of procedure for depositing concrete in the crushed end and side of the old timber cribs as the excavation proceeded, thus causing the leaning piers to right themselves to a considerable extent, is regarded as particularly ingenious. On the whole, it is gratifying to know that Mr. Smith has terminated so successfully the work of transforming the unsteady old piers into substantial, permanent, and symmetrical structures.

THEODORE BELZNER,* ASSOC. AM. SOC. C. E. (by letter).—In his report to Mr. Fisher, Mr. Patton suggested that the material between the bottom of the caisson and the rock might be solidified by "injecting pure (liquid) cement under pressure into the material, thereby converting it into concrete in place." If Mr. Patton or the author made any experiments with this process, it would be interesting to Mr. Belzner.

* New York City.

Mr. Belzner. know the results. The writer recalls a few experiments, made some years ago, in New York City, by forcing grout into sand under pressure. When the material into which the grout had been pumped was removed, it was found that the cement had not been distributed uniformly. The material was lumpy and of no value whatever. Experiments with this process by other engineers may have produced different results.

Mr. Ford. ROBERT H. P. FORD,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Smith has contributed an important and interesting paper, which should prove of value to engineers having to deal with foundations and piers, especially in the large streams throughout the South and Southwest, practically all of which carry large quantities of sediment and are subject to an erratic rise and fall combined with excessive scour, giving opportunities for skill and resourcefulness in work of construction or repair.

The writer was connected with the Engineering Department of the Missouri Pacific System during the period when the steel bent referred to by the author was added to Pier 4, or the "Sick Pier", as it was popularly known, and he speaks from a somewhat personal knowledge of the work that Mr. Smith has done, first as a subordinate and afterward in direct charge of reconstruction, which has been both ingenious and skillful, demonstrating resourcefulness as well as engineering ability of a high order. The writer feels, however, that Mr. Smith has perhaps unwittingly erred in his reference to Mr. C. D. Purdon, an able and accomplished engineer, who was Inspector during the original construction, as well as to others who at various times during the past 30 years have been connected with this work.

Early in 1908, the writer, under the direction of the Chief Engineer of Maintenance of Way, had occasion to take over, among other matters, the work at the Little Rock Junction Bridge, shortly after it had been determined to support the spans on Pier 4 by falsework, preparatory to taking down this pier. It was found at the time that very little information was available that would give a precise technical history of Pier 4, or in fact of any of this work, and, in the judgment of the writer, this was so essential, before a proper conception could be had of the problem, that immediate steps were taken for its collection. It was during this study that the writer became convinced that the authority of Mr. Purdon, as an Inspector, would not have enabled him to prevent unwise methods or improper work.

During the admirable work of Mr. Smith and his assistants, he has had available a fund of information and material which has been developed as a result of long study on this problem by numerous engineers and others, many of the former having recognized standing in

* Chicago, Ill.

the Profession, as well as the effect produced by the various expedients during the time that this bridge has been a source of apprehension; this has also contributed in no small degree to the information bearing on this problem. Mr. Ford.

Until very recently, the engineers' problem on this work has been to temporize with the existing conditions and prepare for an emergency, and, at the same time, to guard, if possible, against a direct failure. How well they have succeeded is perhaps best shown by the fact that nothing did happen, and traffic was maintained. Whether this was due to engineering skill, the forces of Nature, or good fortune, the reader must determine for himself; but to the mind of the writer, the characterization of the matter, up to the time of actual reconstruction, as a "farce comedy", is hardly fair to any of the numerous engineers who were directly or indirectly connected with this work during all this period.

The fact that a large expenditure was made in 1908 to correct this trouble, but which failed to accomplish its purpose, may perhaps be considered as inefficient engineering. The writer, however, hardly thinks this is the case. The records show that the work was done contrary to the recommendations of Mr. Tullock, although apparently approved by the engineers of the Company; but, back of this is the unfavorable comparison between the cost of the work by the method which was adopted and the cost for complete reconstruction, the course which it is believed all concerned would have very much preferred had funds been available.

The situation is not a new one. The Railway Company, like numerous other corporations, doubtless felt that the sum required to enable it to take the safer course would be much greater than for that which seemed to be reasonably sure, although it contained a much greater element of chance. It is a well-known fact that, during the period when the conditions of this bridge were in a serious stage, traffic requirements on the Missouri Pacific System had out-distanced its physical condition to such an extent that the urgent demands for relief were not by any means confined to the Little Rock Junction Bridge. The financial condition of the property has not for years been such as to remedy a great many conditions which its management have at times felt should require attention. In any event, the records seem to show that it was beyond the power of its engineers at any time during this period to do what doubtless in their opinion should have been done, namely, to reconstruct the piers fully and finally, as was ultimately done under the direction of Mr. Smith.

The writer, though fully agreeing with Mr. Smith and his conclusions that, "When defects are discovered in bridge piers, and trouble results, correction should be applied at the seat of the trouble," differs however, with him in his deduction that, "The continuation of the

Mr. Ford. trouble without adequate correction involves constant hazard, which can be avoided by efficient engineering talent".

The writer has every reason to believe that, had this matter been left solely to the judgment of the engineers connected with this work, even from its early inception, or during any of its later periods, the situation would never have been continued.

As is well known, the funds necessary for carrying out projects of this and like character are not always forthcoming, even though the engineer considers that they should be, and this case is no exception. Neither does the writer believe that, even with the numerous recurring changes of management on this property, which has had in its personnel a great many able and efficient men, who were in responsible charge of it, they were so derelict in their duty as to permit a condition to continue that could have been remedied if they had been able. He has reason to believe that the conditions were fully realized, although possibly not to the extent that the engineers may have felt on this as well as other important pieces of work, and, furthermore, that the situation prevailing on the Missouri Pacific in such matters did not differ from that on many other railway properties, where the advancing methods of railroading have caused the need for additional funds which have not always been forthcoming.

Mr. Smith's method of prosecuting the work, as well as his thorough study of the conditions preceding it, have doubtless resulted in saving the property considerable money. The writer, however, is inclined to agree with Mr. Modjeski, in his conclusions (given in Appendix D) that, given the problem as it was, his recommendations were justified. The element of chance in carrying out the work in the manner in which it was done seems to have involved a greater risk, and one which the Railway Company was again willing to assume, thereby introducing an interesting comparison in this particular between the work of 1908, which did not accomplish its purpose, and the work of 1914, which did. Efficient construction and supervision, as well as good engineering, were the elements contributing to its successful conclusion.

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SUBMERGED PIPE WORK AT PORTLAND, OREGON.

Discussion.*

By Messrs. F. M. Randlett, Ben S. Morrow, Clemens Herschel, Kenneth Allen, and R. C. Kellogg.

Mr. Randlett.

F. M. Randlett,† Assoc. M. Am. Soc. C. E. (by letter).—A short discussion of the methods and plant used by Robert Wakefield and Company on the construction of the 30-in. submerged pipe line at Portland, Ore., as described by the author, may be of interest. Mr. A. C. U. Berry, who later was the general contractor for removing the 24-in. and 28-in. and relaying the 24-in. submerged pipes, had charge of the work for Robert Wakefield and Company.

The shore ends of the 30-in. bell-and-spigot, cast-iron pipe were laid in the usual manner. When the first tests of this line were made, three leaky joints were repaired and one cracked pipe was replaced. This part of the work was commenced as soon as the contract was signed, and was ready for the steel pipe connections some time before the dredging was completed and all the steel pipe had been delivered.

The steel pipe was furnished to the contractor, f. o. b. cars, by the City, and the contractor handled the pipe from the cars to the work and stored about 40 ft. of extra pieces in the City yard. This pipe was standard, 30-in. outside diameter, lap-welded steel, galvanized and dipped, made of $\frac{5}{16}$-in. plate, and scheduled to weigh 176.848 lb. per ft. The total shipment, as reported by Robert W. Hunt and Company, inspectors for the City, consisted of 2 081 ft., weighing 464 469 lb., including the flanges. In addition, the order provided for 3 500 $4\frac{7}{8}$ by $1\frac{3}{8}$-in. bolts with hexagonal nuts, weighing 17 730 lb.

* This discussion (of the paper by D. D. Clarke, M. Am. Soc. C. E., published in November, 1914, *Proceedings*, and presented at the meeting of December 16th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Portland, Ore.

Mr. Randlett.

It was contemplated that the flanges would be riveted to the pipe, but it was decided later, after solicitation from the mill, to allow the pipe to be fitted to the flanges and peened, the flanges being then lathe-faced and the holes accurately spaced and bored.

The ball-and-socket joints, made by the John Wood Iron Works, of Portland, Ore., were rotated and tested before leaving the shop, and then hauled and placed by the contractor. Each joint weighed about 5 700 lb.

The method of placing the submerged portion of the line differed from any previously used, in that no special crib, cradle, or tools were required, other than three derrick barges and two common deck barges. Previous lines across the river had been laid with cradles specially constructed for the purpose. By using the plant as described, the work of laying the submerged portion required only 9 days, thereby probably saving considerable expense and causing no noticeable delay to the river traffic.

The work on this submerged part of the line was commenced on the east side of the river. It was necessary to remove a portion of the O. W. P. Dock in order to lay the pipe on the incline just east of the harbor line. After placing the inclined section and two additional lengths west of the bottom of the slope, the procedure was as follows: One derrick held the westerly end above the water, while three, and occasionally four, lengths of pipe with one flexible joint (already bolted together) were bolted to it; the derrick then lowered the pipe until only the westerly end was above the water. The total lift varied from 8 to 12 tons before submerging.

The slings were marked and care was taken while lowering that the flexible joints were not allowed to deflect more than 11 degrees. The rear slings were then slipped, and the operation was repeated. In lowering the pipe it was necessary to permit water nearly to fill the pipe, as it was almost buoyant, and it was desirable to keep the strain on the three derricks as nearly alike as possible.

The dredging was done by the Water Department with a 20-in. hydraulic suction-dredge, owned and operated by the Port of Portland Commission, and the *Titan,* a large dipper-dredge owned by the Pacific Bridge Company. The suction-dredge excavated most of the material, the dipper-dredge being used only when it was necessary for the suction-dredge to be used by the Port of Portland on lower river work.

The only delay was caused by the failure of the steel pipe to arrive when expected, which resulted in an extension of the time set for the completion of the work. All parties concerned in the work co-operated in every possible way, and the various parts progressed without delay. Progress on the West Side, under the Southern Pacific and the Oregon Electric Railway tracks, was slow, but no interruption to traffic occurred.

Mr. Randlett.

While placing the pipe in the river, one of the slings broke, damaging one ball-and-socket joint and the ends of two lengths of pipe. The damage was quickly repaired by the contractor, and no delay was caused thereby.

The valves on this line are extra heavy, 30-in. Rensselaer, double-disc, gate-valves, with by-pass and indicator. There are two 4-in. Lunkenheimer, relief valves on each end of the line at the gate-chambers, and set to blow off at 135 lb., which is 15 lb. in excess of the average working pressure at the points mentioned.

The gate-chambers are of concrete, with reinforced concrete top and standard manhole covers. There is a standard cast-iron gate-box casting in the concrete top over the operating nut of each valve, so that the latter may be operated from the street with the regulation gate-key.

A portion of the steel pipe on the East Side of the river lies on a gravel fill between East Water Street and the harbor line, and is not completely covered by gravel. During the high-water stage of the river this pipe is entirely submerged, but is exposed at all other times. A recent examination of this pipe shows absolutely no indication of pitting or rusting, although the dip coat is practically gone, the galvanizing being exceptionally clear and bright.

Tests of this pipe line in May and June, 1914, showed a leakage of about 1 000 gal. per 24 hours.

Mr. Morrow.

BEN S. MORROW,* ASSOC. M. AM. SOC. C. E. (by letter).—A brief description of the method and equipment used in carrying out this work may be of interest.

In connection with taking up and relaying the 24 and 28-in. lines the dredging work was naturally divided into three operations: (1) Removing the cover over the 24-in. line; (2) excavating a trench for the new location of the 24-in. pipe to a depth of 38 ft. below low water for the entire distance between harbor lines; and (3) removing the fill over the 28-in. pipe.

The work, in the main, was done by a dipper dredge capable of handling 1 400 cu. yd. of sand and gravel in an 8-hour day (sand and gravel constituting practically all the dredged material). This dredge could work to a depth of 44 ft. below water surface, and the stage of the river was such that during the entire operation it was possible to dig to the required depth of 38 ft. below low water. An effort was made to use a floating derrick operating a clam-shell bucket in uncovering the 24-in. pipe, but it was found that the bucket was damaging the pipe, and its use was discontinued.

The average cost of dredging and removing the material was approximately 12 cents per cu. yd.

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* Portland, Ore.

Mr. Morrow.

The removal of the 24-in. line was begun on the west bank of the river, and the pipe was disconnected above the water level. The first six lengths were taken up by a land derrick. Slings were put around the pipe by a diver, and care was taken not to rotate any flexible joint through a greater angle than 15°, which was the maximum possible movement for the ball-and-socket joint. By raising a pipe a few feet and slinging it to the bents on which the land derrick operated, and then raising the next pipe a few feet and repeating the operation, the pipe was taken up out to the point where the two floating derricks could take hold.

Very early in the course of this work it was found that, when a flexible joint was rotated through a greater angle than the allowable 15°, the joint where the steel pipe was leaded into the cast-iron flange began to give, and under a slight strain the pipe would pull out entirely. This was due to the fact that the lead ring in the flange was not as deep as required by the present standard. On examination it was seen that the pipe pulled on the flange without damage to itself or the ball-and-socket joints. The contractor took advantage of this condition, and, putting slings on two or three pipes at a time, brought it up piecemeal, without doing it any material damage.

This pipe was taken up by a crew consisting of a foreman and seven laborers, a diver and helper, and two floating derricks, of 45 and 25 tons capacity, respectively, at a cost of $1.70 per ft. The pipe was taken up to a point 100 ft. inside the east harbor line, where it was disconnected under water by a diver.

When originally laid this pipe had been dipped in a mixture of pure-graded refined California asphalt, fluxed with a high-grade natural liquid asphalt heated to 280° Fahr., and, although it had laid under the river for 16 years, the coating was absolutely intact and, with one exception, the plate was in as good condition as when laid.

After the pipe was taken out, the flanges were removed and the pipe thoroughly sand-blasted. The lead rings on the flanges were made deeper and the pipes were redipped in a mixture of 80% "Manco" pitch and 20% California asphalt, "F" grade, at a temperature of 300° Fahr. This mixture has a melting point of 175°, and is entirely flexible at 32 degrees. It shows no tendency to become soft and sticky at 100° Fahr.

The ball-and-socket joints were taken apart and the balls machined, in order to remove all pits and rust spots. On being re-assembled the joints were tested to 300 lb. It was then found that after being subjected to test pressure the flexible joints could not be rotated until the ball had been forced back into its original position, for, under test pressure in the shop, the ball, although a perfect sphere, would compress the lead as much as $\frac{3}{16}$ in. and lock itself. This would tend to bear out the theory that, in laying a flexible joint line, a tighter

line can be obtained by stretching it by bolting on a head and applying pressure at frequent intervals in the course of laying. Mr. Morrow.

The 24 and 28-in. pipes were 100 ft. apart for the greater distance between harbor lines, but at the east line the two pipes were only 10 ft. apart. For the new location of the 24-in. pipe it was necessary to dredge 18 ft. below the 28-in. line and still maintain the pipe in service. This was accomplished successfully by driving a pile bent over the pipe every 16 ft. and supporting the pipe in cable slings.

In relaying the 24-in. pipe the initial connection was made in 20 ft. of water by a diver, and in such good time that it was proposed to bolt up six lengths of pipe on the barges and have the diver make an under-water connection in each case. The fourth connection under this system required 4 days, and it was then decided to make the connections above the surface. By using graduated slings on a bridle it was possible to lay the pipe without straining the flexible joints which were placed at intervals of 40 ft. The diver released the slings as the pipe reached its final position in the trench.

To handle this pipe successfully it required three floating derricks, operated side by side, and one small barge derrick for shifting the pipe. This method of laying proved very successful, and a crew, consisting of one foreman, nine laborers, a diver and helper, with four floating derricks, laid this pipe from harbor line to harbor line at a cost of $2.04 per ft.

Tests on this line as relaid showed a leakage of 2 100 gal. per day, 10 months after the completion of the work.

Removal of 28-In. Line.—The removal of the 28-in. cast-iron pipe, after the completion of the 24-in. pipe, was an easy matter. Due to the type of joint, this pipe can be hauled out of the water like a cable—the lead and the cast-iron yoke lock the pipes together so securely that, after the first lengths had been broken away and raised to the surface, the diver was not needed to put on the slings, as they could be dropped down under the pipe to a point below the ball and the 45-ton floating derrick supported the load while each pipe was disconnected. It was necessary to melt the lead joint with a blow torch as the flange on the socket of this pipe extends several inches beyond the center of the ball. Melting out the joint was slow work, and the time saved by the method used in raising the pipe was lost in disconnecting on the barge.

The cost to the contractor of taking up this pipe was $2.00 per ft. for a foreman, seven laborers, and two floating derricks.

This pipe had been laid for 20 years, yet was in very good condition when taken up. A slight cut taken off the balls will put it in condition for relaying at some future time.

Such figures as to the cost of removing and relaying these pipes as are given are based on work done between harbor lines under the

Mr. Morrow. most favorable conditions, with no sudden or material changes in the stage or currents of the river, which was a few feet above low water; nor do these figures take into consideration the cost of the work outside of harbor lines, which was about three times as expensive as the work on the river. The expense to the contractor for breakage to the cast-iron pipe and yokes, due to the method of taking up the 28-in. pipe, is not considered in the cost figured for this work.

Mr. Herschel. CLEMENS HERSCHEL,* M. AM. SOC. C. E. (by letter).—When it is considered that any riveted or screw-jointed pipe line, of whatever diameter, has a certain radius to which it may be bent without causing leakage of the contents, it raises the question why this kind of pipe should not wholly replace hinge-jointed pipe lines.

If such a steel pipe line, though even of tunnel dimensions, be loaded on the exterior, or interior, or both, so as to give the combination a specific gravity slightly greater than 1.0, and the front end be then closed, water-tight, the pipe may easily be hauled across on the natural, or on a prepared, bottom of the river, to which it will fit itself by virtue of its inherent flexibility.

Weighing little or nothing in the water, it exerts little or no pressure on the bottom on which it slides; and, without material pressure, offers no material frictional resistance.

Heavy pipe, or pipe full of water, may also be hauled across on the bottom. Sixteen lines of 12 and 18-in. flexible-joint cast-iron pipes have been thus hauled across a 1 000-ft. channel, at Vancouver, B. C.;† and quite recently the technical press has contained accounts of the hauling of about a mile of pipe out from a sand beach into deep water, off Tampico, Mexico, to enable tank steamers to load with oil.

In 1895 the writer laid seven lines of 18-in. screw-jointed pipe (the largest size of welded pipe then made) by the method described, in a trench dredged for the purpose, across the Passaic River at Belleville, N. J., at a narrow crossing that was constantly impeded with passing vessels. Flexibly jointed pipe could not have been laid at this place from floats or scows.

A flagstaff set on the forward end indicated where that end was at any moment; and the rear end was added to, and extended backward, on a kind of launching way, as the pipe passed into the water from that end.

During the passage of the forward end across on the bottom of the river, filling the pipe with compressed air furnished proof that there was no leakage; and for 19 years a Venturi water meter, set in this line of pipes, on each shore, has proved that they do not now and never have leaked, though under some 350 ft. head. On the other hand, it is doubtful if any form of flexible-joint pipe ever laid will

* New York City.

† The method is described in *Engineering Record*, September 6th, 1913.

stand, or would have stood, such a test equally well, it being remembered that water under 350 ft. head will cut out lead very rapidly if a leak is once started. Mr. Herschel.

A 6-ft. pipe was laid under the Hackensack River by this general method some years later. The fact that it broke apart at one point at the bottom of the pipe, shortly after it was laid, is no criticism of the general method of laying such pipe, here described, but only of the special method of loading the pipe in this instance. The break was immediately repaired, and the pipe is to-day doing good service.

The loading must be designed so that it will not affect the natural flexibility of the pipe, which is far greater than one would suppose. Speaking from memory, a riveted 4-ft. pipe can be made to assume a curvature of 1 000 or 1 200 ft. radius, without causing the pipe to leak; and such a radius is easily attainable in a river cross-sectional profile; or the pipe may be built on a curve following the profile across the river selected, a special curved launching way being used when hauling the pipe across.

In the writer's opinion, the general method here described could be used advantageously in the construction of subaqueous tunnels; but this, maybe, "is another story".

KENNETH ALLEN,* M. AM. SOC. C. E.—The speaker would like to ask Mr. Wiggin whether, with such a finely machined surface in the joint, by which any bending load is transmitted directly through the metal of the pipe, it would not have been practicable to use, instead of lead, some composition having a coefficient of expansion by heat of zero, and thereby do without caulking? Mr. Allen.

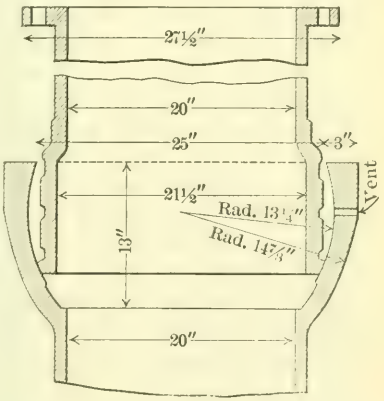

FIG. 5.

At Atlantic City, there was a 20-in. submerged pipe crossing Beach Thoroughfare and a 12-in. pipe crossing Inside Thoroughfare, both tidal streams. The former consisted of three lines of bell-joint pipe, two of which were nearly laid before the speaker took charge of the work. The third line replaced an old 20-in. line, and was similar, except as to the details of the joint, which was leaking and in a generally bad condition. Fig. 5 shows the new joint. One difficulty experienced with this joint was that when, on laying, it was deflected sufficiently, one of the projecting rings on the spigot end came so near the bell that there was not sufficient clear space

* New York City.

Mr. Allen. for effective caulking by the diver, in case of leakage. This difficulty was obviated by having the projecting rings on the inside of the bell, as in the Portland design.

In taking up the old line, several joints were pulled apart by the tension in lifting. The new lines were laid on cast-iron cradles resting on sills. Each sill was bolted to two piles, forming a bent, which were afterward cut off below water. The length of the crossing was about 240 ft., and the bents, shown on Fig. 6, were 12 ft. apart. The pipe was caulked in sections of six or seven lengths above water. Each section had a flange at each end and was lowered a certain distance, on a given signal, by a man at each bent with a block and fall, until it rested on the cradles. The flanged end was then bolted to the previous section by a diver, the depth being, according to the speaker's recollection, from 12 to 16 ft., depending on the tide.

The pipe, bought as an emergency measure on the default of an earlier contract, cost $87.50 per ton. The cost of removing the 240 ft. of old line of submerged pipe was $2.90 per lin. ft., and that of laying the new pipe, including dredging, $7.90 per lin. ft. The total cost of taking up the old main and relaying the new one in its place was $9 762.

The 12-in. line was made up and caulked on shore in sections of three lengths with a Walker "Half Ball Joint" on each end, except at two places where Walker "Universal Ball Joints" were used. Each section was then lowered from a scow to a dredged trench and loosely bolted to the previous section. The line was laid in a slightly zigzag position to permit a reasonable amount of settlement. After allowing a certain time for the pipe to find a firm bed, the bolts were tightened up by a diver. The "Universal Joints" were placed where further deflection would be most probable. The maximum depth of water in this case was nearly 30 ft.

After completing the 316 ft. comprised in the crossing, the leakage was tested by placing a ¾-in. Trident meter on a pipe by-passing the valve at the supply end of the crossing. The small leakage which occurred at first was finally entirely stopped by a diver. It is doubtful whether the line has remained without any leakage, but the speaker had no further opportunity of testing this.

The total cost of this line was $1 799.40, or $5.69 per lin. ft.

Mr. Kellogg. R. C. KELLOGG,* JUN. AM. SOC. C. E.—In regard to the subject of caulking pipes under water, the speaker has had some experience with two gas mains laid across the Harlem River, in the City of New York. One of these was a 20-in. main, laid in 1890, from the foot of 129th Street and Second Avenue to Lincoln Avenue, The Bronx. The other

* Brooklyn, N. Y.

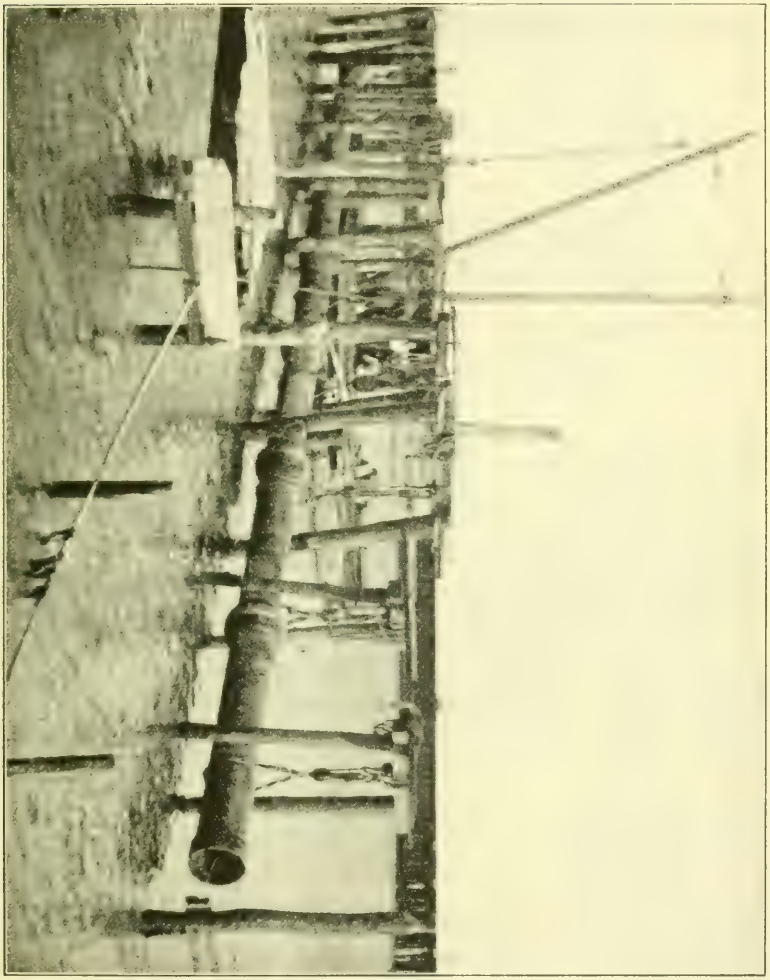

FIG. 6.—LAYING PIPE AT ATLANTIC CITY.

was a 24-in. line crossing the river between 135th Street, Manhattan, and Mott Avenue, The Bronx. Mr. Kellogg.

The method of laying these lines was practically the same as that described in the paper and in some of the discussions, that is, by making the joint above water and lowering the pipe to the bed of the river with the derrick. There was this difference, however, the joints were made up of yarn and poured lead on the scow derrick and, after being lowered into the water, were re-caulked by divers by hand. This was possible because the pipes were permitted to rest on heavy timber blocks laid immediately back of each hub. The pipe had Ward flexible joints and the hub was of such a diameter that the spigot could be entered rather freely.

In the article* by the late Emil Kuichling, M. Am. Soc. C. E., referred to in the paper, a page is devoted to illustrations of joints used in submerged pipe work. Fig. 13 on that page shows the joints used on these gas lines.

These mains have shown no leakage. The 20-in. line has been broken three times, to the speaker's knowledge, by the dredging operations necessary to keep the channel in the Harlem River to the depth prescribed by the Government.

When these mains were laid, the Gas Company took chances on this very contingency, because they were placed in a shallow trench formed by blowing off the top surface mud with a jet from a hose.

One speaker has referred to the effect of the blow of the hammer in caulking joints in pipes, with special reference to the distance back from the face of the joint in which the impact of the blow is effective in compressing the lead. For 7 years the speaker has used hot lead and lead wool in caulking joints in gas mains, and in that time, has had occasion to examine the effect of the blow of a hammer on hot lead and also on the comparatively new product, lead wool. Several joints of each kind have been examined by sawing through the joint, thus obtaining a section exposing the yarn and lead. Where hot lead is used, in no case has the compacting effect of the blows of the caulking hammer extended back more than $\frac{1}{2}$ in. from the face of the joint. Furthermore, when poured into a joint, hot lead contracts on cooling, and draws away from the inside face of the bell, leaving a space of from $\frac{1}{64}$ to $\frac{1}{32}$ in. in width.

In caulking with lead wool, the process is cumulative, that is, the joint is made by driving in successive strands of lead until the caulking space is entirely filled. Cross-sections of joints made with lead wool show a solid piece of lead from the face of the joint to the yarn, and the first strand driven in has the additional effect of compressing the yarn considerably. The speaker has laid gas mains

* *Engineering and Contracting*, April 15th, 1914.

Mr. Kellogg. under the Harlem River, in which the joints were made exclusively with lead wool.

The discussion of the paper by members connected with the New York Board of Water Supply and engaged in constructing The Narrows Siphon, is very interesting; the form of joint adopted for that work shows great ingenuity, and is a credit to them.

It is unfortunate that the depth of water, tidal currents, and commercial tonnage passing through The Narrows, together with its exposed condition, made it impossible to have the work done by divers. The speaker would like to suggest, however, that there is a diving apparatus now manufactured for which the makers claim a possible diving depth of 200 ft., and investigation of its merits might prove of interest in case the present scheme proves unsuccessful under working conditions.

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PAPERS AND DISCUSSIONS

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DEPRECIATION AS AN ELEMENT FOR CONSIDERATION IN THE APPRAISAL OF PUBLIC SERVICE PROPERTIES.

Discussion.*

By Messrs. James D. Mortimer and Joseph Mayer.

Mr. Mortimer.

James D. Mortimer,† M. Am. Soc. C. E. (by letter).—Mr. Grunsky is quite correct in asserting that the inclusion of allowances for depreciation in operating expenses, when testing for reasonable rates, is essentially different from the amortization or return of invested capital to the proprietors of a public utility. The obligation to make replacements of worn out, inadequate, or obsolete plant runs concurrent with the operation of the utility. These facts were pointed out in the writer's discussion on the Report of the Special Committee on Valuation of Public Utility Properties, to which reference is now made.‡ The indeterminate nature of the problem of future life of a particular unit of equipment was also pointed out, and the similarity of the estimates of depreciation allowances to those of the life insurance actuary was indicated.

1.—After a unit of equipment has become part of an operating plant, the essential difference in value between that of an old and of a new unit lies in the probability of the former requiring earlier replacement or abandonment than the latter. The old unit may serve its purpose just as well as would a new unit of the same or of an improved type. The difference in relative value must accordingly

* This discussion (of the paper by C. E. Grunsky, M. Am. Soc. C. E., published in November, 1914, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

‡ *Proceedings*, Am. Soc. C. E., for April, 1914, p. 1174.

Mr. Mortimer.

be measured by some factor that has to do with the time of its future replacement. This factor can be most conveniently measured as that reserve which will insure the replacement of the unit at the end of its probable life time. This thought has led to the development of the expression, "Replacement Insurance", to cover the concept ordinarily described as "Depreciation."

2.—If a utility has accrued an adequate reserve for replacement insurance and the same is properly recorded on the books of the corporation, the property value or capital for purposes of rate-making should contain no deduction for depreciation in whatever manner it may be measured. If, however, the corporation fails to record this liability on account of replacement insurance and disburses the amount allowed in earnings therefor, in the form of interest and dividends on its capital obligations, there apparently results a withdrawal of capital from the utility, and the fact that "Depreciation" should not be deducted is not so clear.

3.—Regulating commissions and Courts rarely have to deal with cases involving public utilities in which there has been a complete reservation for replacement insurance. The problem usually met covers the case where the reservation has been only partial, or, more frequently, where the reservation has not yet been begun. The cases contemplated in Mr. Grunsky's hypothesis, accordingly, are the exception rather than the rule.

4.—Although every public utility corporation has the implied obligation to make replacements of worn out, inadequate, or obsolete units of plant, as they become necessary or advisable, it is not always readily evident that such corporations possess the financial ability to fulfill such obligations. If the assets of such a corporation should be mortgaged for the purpose of securing an issue of bonds, many cases arise where the obligation to make replacements cannot readily be enforced against the bondholders. Capital stock of such corporation is usually issued "full paid and non-assessable", and the obligation to make replacements cannot in such cases be enforced by assessment of the holders of capital stock. In view of these legal obstacles, it is not surprising that regulating commissions and Courts have been inclined to deduct the contingent obligation for replacement from the estimated cost new of the property, when considering valuations for rate-making.

5.—When the property constituting a utility is transferred from one stock corporation to another, or to a municipality, the transfer presumably carries with it the implied obligation of future replacements of physical plant. The purchaser, in assuming this obligation, will either require that it be shown by assets other than physical property or deduct the amount of such obligation or liability from the purchase price that would obtain were there no such obligation.

This would seem to indicate a possible difference in value for the two purposes of rate-making and sale. Mr. Mortimer.

6.—The rate cases which have been before the United States Supreme Court have invariably been presented on the theory of confiscation, as this is the only basis on which such cases may be brought before the Federal Court. The idea of confiscation contemplates the acquisition of the whole or a part of the property. Thus, rate cases before the Federal Courts automatically become purchase cases, and, in view of this fact, the dicta of the Supreme Court are essentially applicable to valuation for purposes of sale. In such cases, however, there appear to be important elements of value, all of which are referred to in the Smythe *vs.* Ames Case, but which have obtained only slight recognition in the actual monetary measurement of value.

7.—As value for purchase and sale may depend on earnings, and earnings in turn may depend partly on rates of charge for service supplied by the utility, it is not unnatural that regulating commissions should have endeavored to appraise the capital investment devoted to public use by some method which would be independent of current earnings, that is, independent of the rates of charge for service. Although the definitions of principles of valuation are of great importance, from the standpoint of law, economics, and social equity, the making of reasonable rates of charge for service depends on many factors, of which property valuation is only one. The final test of reasonable regulation cannot be formulated with mathematical exactness, but will be judged by the attitude of the proprietors of the utility affected toward the investment of additional capital. The reasonableness of regulation may be approximately ascertained without an attempt at valuation, as is shown by the present-day situation of the steam railway carriers.

8.—It seems readily evident that the regulation of public utilities may be justified in law without introducing the doctrine of "principal and agent", to which the author makes incidental reference, and which has achieved some popularity with economists of a certain class. This theory is wholly unnecessary to a proper definition of all the principles of regulation, and unfortunately possesses a slant in a direction toward which there is no good reason for regulation to tend.

JOSEPH MAYER,* M. AM. SOC. C. E. (by letter).—The author gives two definitions of depreciation. He says, under "Depreciation", that it is to be measured "by a comparison between the probable life, the expectancy, and the cost of replacement at the expiration of the term of the article's actual serviceability". Mr. Mayer.

Under "Remaining Value" or "Present Value", he says:

"This is the present worth of any article dependent on, and to be computed from, the three elements: (*a*) the probable useful life of

* Montreal, Que., Canada.

Mr. Mayer.

the article new; (*b*) its probable remaining life; and (*c*) the cost of replacement, which when prices are not subject to change, is the original cost less the residual value of the article when it goes out of use."

When prices are subject to change, as is nearly always the case, the author defines "Replacement Cost" by: "The cost of effecting a change from the worn-out part to a new part of equal service value." He evidently means of equal service value with the worn-out part when it was new.

The author does not state how the computation of the remaining value is to be made, but one would naturally infer that, when there is no residual value, the remaining value is the cost of replacement multiplied by the probable remaining life and divided by the probable useful life of the article new. Or:

$$\text{Remaining value} = c\,\frac{b}{a}.$$

The author evidently believes that interest has nothing to do with the case, but, later, he mentions obsolescence and inadequacy as influencing the probable remaining life. The definition contains two serious errors. The article has a certain annual service value, a, during an estimated number, n, of years. If i is the rate of interest, if the residual value of the article is zero, if a and the value of the new article are constant, if $f = \frac{100}{100 + i}$, and if the annual service values are received at the end of each year, then the total service value, S_n, of the article, which is the sum of the present values of all its annual service values, is

$$S_n = af + af^2 + \ldots\ldots + af^n = af\,\frac{1 - f^n}{1 - f}.$$

If the probable remaining life of the old article is r years, then the total service value of the old article is:

$$S_r = af\,\frac{1 - f^r}{1 - f},$$

and the ratio of the two values is:

$$\frac{S_r}{S_n} = \frac{1 - f^r}{1 - f^n}.$$

Therefore, if the given assumptions are fulfilled, it is not necessary to determine the constant annual service value of the article in order to find the ratio of the values of the old and the new articles. According to the author's definition,

$$\frac{S_r}{S_n} = \frac{r}{n}.$$

Mr. Mayer.

He would make the remaining value of an old article, which has a probable remaining life of 20 years and a probable life when new of 60 years, of one-third as much value as the value of the new article; but, with 6% interest, as $f = \frac{100}{106}$, the writer obtains

$$\frac{S\ 20}{S\ 60} = \frac{1 - \left(\frac{100}{106}\right)^{20}}{1 - \left(\frac{100}{106}\right)^{60}} = 0.710.$$

The real value of the old article, with the assumptions made, is therefore 71%, and not one-third of the value of the new one. The depreciation is 29%, and not 66⅔%, as given by the author's definition.

The assumptions agree with those made by the author. They agree, however, but seldom, if ever, with the facts. Most articles lose in efficiency of service with age, and the cost of their maintenance generally increases toward the end of their useful life, until the net revenue, a, derived from the use of the article, becomes zero, when, provided it has no residual value, it will be discarded. If it has a residual value, it will be discarded sooner and used for another purpose. Therefore, the annual service value, a, is generally not a constant, and may suddenly change when a much cheaper equally serviceable or useful article for the attainment of the same end comes into the market. Obsolescence, therefore, affects not only the probable remaining life, but also the annual service value, of an article. Though the appearance of a cheaper article of equal usefulness does not modify at all the material usefulness of the old article, it does reduce the value of this usefulness as its value depends on its cost.

This applies also to an obsolescent plant as a whole; its remaining value as a plant does not exceed the cost of a modern plant of equal usefulness. The land of the plant, together with its parts, however, may be more valuable than the plant; it will then probably be dismantled.

The value, S_n, of an article is measured by its market price. The equation,

$$S_n = a\, f\, \frac{1 - f^n}{1 - f},$$

$$\text{gives } a = \frac{S_n}{f}\, \frac{1 - f}{1 - f^n},$$

$$\text{where } f = \frac{100}{100 + i}.$$

i is here the annual profit, and a is the cost per annum of the article. The use of an article is only justified if its annual usefulness exceeds

Mr. Mayer.

the annual cost, a; in this case its value in the plant is equal to its cost.

For an article, S_n is the price, which can be easily ascertained, and therefore a can be found when n and i are known. For a business enterprise, there is generally no known market value, S_n, but its net revenue, a, can be ascertained. If a is variable, then

$$S_n = a_1 f + a_2 f^2 + - + (a_n + R) f^n$$

where a_1, a_2, . . . a_n, are the succeeding annual revenues, and R is the residual value of the plant when it is abandoned.

Besides depreciation, the author discusses appraisal. Appraisal means the finding of the value of a thing. What is wanted is the fair value, or the value correctly ascertained. The Courts say that the fair value is the present value, and that investment or original cost and additions of cost to date, cost of reproduction new less depreciation, past, present, and prospective revenue, market value, and possibly other factors, must be considered in ascertaining this value. They maintain that there is only one fair present value, and not different values for different purposes.

The value of a thing, as defined by economists, and, on the whole, accepted by the highest Courts, is the quantity of other things it will buy, when the buyer and seller are good judges of value and in a position to act in accordance with their judgments of values.

From the standpoint of the investor, a public utility is a tool to secure a net revenue. The investors or possible buyers of such a utility judge its future net revenue from its recent and present net revenues and the causes of changes in the future.

If the utility is subject to regulation, its net revenue is influenced to a large extent by this regulation, and its future value depends on the nature of the regulation. The investors are mostly convinced that the Courts will protect present values of past investments, and will not permit their confiscation by a change in the regulation.

The present fair value of a public utility, therefore, is given by a correct estimate of the present value of the future net revenues which would be obtained with the present regulation.

Any legal regulation must pay the past investors a fair return on the present value, or it must give them the present value.

The author says:

"'Fair Value', then, is necessarily based on proper and reasonable investment which may be ascertained from cost records, and, when cost records are not available, is usually estimated by the 'cost-of-reproduction' method."

Values change with conditions and laws. The values of products supplied by many competitors approach cost of production, which includes cost of labor and of use of capital, rent, taxes, and other

incidental expenses, and very variable profits. The profits of each producer, together with the other costs which he incurs, must result in such prices of the articles produced that they can be sold in competition with similar articles offered by other producers. The average profits in all industries, but not the individual profits of different competitors in them, tend, with free competition, toward equality. With free competition, very different profits are obtained by the creators of individual industrial plants, and any estimate of value based either on actual costs, or cost of reproduction, assumes uniform profits for the creation of plants or business enterprises, whether they are planned, built, and managed by the highest industrial talents or by mediocrities. It is, therefore, necessarily erroneous, even if it is made with the utmost care and without any other error. Mr. Mayer.

Recently the bondholders of a nearly new railroad, 90 miles long, in Western New York, proposed to dismantle it because it could not be operated without loss. This railroad is worth less than one-fifth of its actual cost or its cost of reproduction. Such extreme cases are rare, but very large differences between values and either actual cost or cost of reproduction with uniform profits are the rule and not the exception. Natural monopolies existed for a long time without regulation. High prices and excessive profits often resulted therefrom. This produced a rush of capitalists to acquire such monopolies, and many enterprises were started long before they could be made profitable. The investors expected to be repaid for early losses by later large profits. If they succeeded, the public had to pay for the unnecessary losses of premature enterprises. Competition for the acquisition of the monopolies did exist, but excessive prices, nevertheless, often resulted.

This led to the regulation of the prices of monopolized products. The general endeavor was to establish such prices as would give the same return on invested capital in monopolized as in competitive industries. Before regulation existed, each enterprise obtained what profit it could get, and the profits on the actually invested capital differed very widely, according to the intelligence and skill of the promoters and managers of the enterprises. The values of the enterprises which arose under such conditions are neither equal to the investment nor to the cost of reproduction less depreciation. They are governed by estimates of the present values of the future net revenues, and they can be ascertained only by such estimates, and not by either of the two methods advocated by the author. Neither of these methods is sustained by the higher Courts, and no sane buyer or seller would follow either of them. It is often arbitrarily assumed that, because the actual future revenue is dependent on the future regulation, revenue cannot be used as the basis of the valuation for the purpose of rate-making. This is bad logic. As the present values,

Mr. Mayer.

protected by the Courts, are those which would prevail with the present regulation, they are independent of the future regulation as long as this protection lasts. As the properties, in the future, unless assisted by the public, must earn a fair return on their present values, the future regulation depends on the present values.

The fair return may be much smaller than the present one, if the present owners are given safe securities of the same market value as their properties. Keeping promises is the foundation of all honest business, and this is what the Courts insist on. The method of valuation advocated by the writer is in substantial agreement with the decisions of the highest Courts. Both the methods defended by the author, if adopted, would revolutionize all existing values of public utility companies. That which he prefers would give for old properties results farthest from the actual values. This method is not at all defined by him, as he would fix arbitrarily what part, if any, of the development costs would be included and what part of deficient or excessive past income would be considered.

Mr. Grunsky endeavors to describe and improve a hopelessly confused industrial system which has arisen from empirical attempts to establish, by means of regulation, just prices of monopolized products. He advocates the principle that the private owners of public service properties should receive remuneration in proportion to the investment.

The author defines Investment as "the aggregate of the reasonable and proper expenditures which have been made to render the property in question efficient for the purpose for which it is intended." From the standpoint of the public the purpose of a public utility is to give a product or service, of the desired quantity and quality, at the lowest possible price. The reasonableness of any expenditure depends on its efficiency to secure its ends, or on whether its usefulness justifies its cost. This depends mainly on ascertaining correctly the difficulty and importance of each part of the work, fixing an adequate compensation in proportion thereto, securing the best men obtainable for it, in judging the merits or the usefulness for their purpose, of the various materials needed, and selecting the cheapest when all effects are considered.

There are only two ways of securing high efficiency of expenditures, or, what amounts to the same thing, a considerable amount of reasonableness: One is to put a capable man at the head of an enterprise, with full power to select the proper means for prescribed ends, and to make him responsible for results; the other is to make the compensation of the independent owners proportional to the efficiency they secure. These independent owners may then be trusted, with little supervision, to select fit men, but they cannot be trusted to do this if their compensation is in proportion to their investment. In

this latter case, the risk is with the public; full control must go with the risk. This means public management of promotion, construction, and operation, or, in effect, public ownership. If there is private management of promotion, construction, and operation, no outsider can judge correctly the reasonableness of expenditures. He cannot possibly secure a high degree of reasonableness; that can only be obtained by the actual manager in charge. The manager, therefore, must be appointed and discharged by the representative of the parties most interested in the efficiency. This party is, with the author's method, the public, and it must appoint the managers. Mr. Mayer.

The author's method of controlling and rewarding private owners of public utilities is impracticable and unreasonable. Therefore, it is impossible to make it consistent and definite. Though he endeavors to secure for the owners a fixed return on the investment, he is obliged to define the investment so that it is not the amount of money honestly spent by the investors, but something else, which cannot be ascertained correctly with private management. He is also unable to define when inadequate earnings to secure the desired rate of interest on the investment, either because the regulation was imperfect or because even without regulation the property could not earn the interest allowed, shall entitle the owner to recover past losses by future earnings.

If an earthquake or a flood destroys part of the investment, the author leaves it in doubt whether or not the destroyed investment is entitled to future earnings.

The decision of all these questions is left in the hands of authorities elected by the consumer, subject to control by the Courts.

The contract between the owner and the consuming public is so extremely indefinite that nobody can know what is a fair construction of it. Under such circumstances, justice becomes impossible, and innumerable lawsuits can only be prevented by making them so expensive that both parties will submit to a very large amount of injustice rather than go to law. Capital, therefore, can be secured only by promising an excessive rate of interest. The investor in such enterprises is practically in the position of a lender of money to a man with very doubtful credit; he is not sure whether he will obtain the promised interest or will ever get back the principal. Capital can be obtained at a moderate rate, if the public takes all the risks and adopts full public ownership wherever a definite contract, leaving the risks with the private owner and giving him only a fair return, is impracticable.

For this purpose the value of such utilities as are best publicly owned and operated must be determined by estimating the present value of their future earnings with the present method of control, provided this control cannot be shown to be illegal and unfair. New bonds, with a moderate rate of interest, can then be issued to pay with

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their proceeds to the present owners the values of their properties. The properties should then be managed by general managers with full control, responsible to boards of directors, who prescribe the ends to be attained, watch the efficiency of the manager, and are responsible to the public. As money can be obtained on bonds under normal conditions at from 4 to 5%, the change to public ownership would secure, with proper valuations and good public management, a large saving to the consumers.

In all his calculations the author neglects the various risks of the owners, which are very different under different circumstances. As the values of such properties are much affected by these neglected risks, his calculations of values are necessarily unreliable. He also constantly fails to distinguish between the widely different values under different laws and preponderant ideas of justice.

If a regulation were established which would follow consistently the principle of a uniform return on investments, without the various exceptions which the author is obliged to make in order to avoid absurdity, then the values would be in proportion to the investments; but entirely different laws govern values under free competition, under natural unregulated monopolies, and under any other kind of regulation. When one economic system is abandoned and another is introduced which produces entirely different values, the only just way of ordering the transition is to give to the owners of preceding enterprises the values which have arisen under the old system, or to offer them securities which they are willing to accept as equivalent.

The author in this and his previous paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates",* to which the present paper is a supplement, has Californian laws and conditions mainly in view, and considers principally those utilities for which public ownership is the only practicable method of regulation; but he does not limit the application of his theories to such cases.

He may object, to the method advocated by the writer, that for new properties it is impracticable, as the future revenues which a new property will be able to secure with the present regulation, cannot be estimated, and that the actual cost of such a new property is, therefore, the best available measure of its value. With a regulation adopting the principles here and previously advocated by the writer, which proposes to secure equal average profits in monopolized and competitive industries, it is not necessary to know the value, but only the investment, in new enterprises.

The profit of an enterprise is the annual return on the capital invested. When a new regulation is introduced, the owners of old

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 828.

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enterprises must be given the values that have arisen under the old regulation, therefore these values must be considered as the investments in the old enterprises; and, for new enterprises, the actual investment must be taken. The annual net earnings of a monopolized industry must be compared with the investment thus ascertained in order to obtain the average profit of this industry. This average profit should be the same as the average profit of competitive industries of about the same risk.

It will not be practicable to ascertain the average profit in private firms and partnerships, but the average profits of stock companies engaged in competitive enterprises can be ascertained with the proper regulation necessary to secure the interests of minority stockholders. These enterprises are most nearly like public service properties and, therefore, are best used to determine the average profit to be allowed to monopolized industries. After the investments in monopolized enterprises have been thus ascertained, they can be correctly kept up to date by a moderate amount of supervision.

For this purpose the author's definitions of Investment and Depreciation must be modified. The aim of this method is to secure in the future equal average profits on the actually invested capital of competitive and monopolized industries.

The proper definition for Investment, therefore, is: The capital invested is the aggregate of the actual expenditures which have been made in order to render the property in question efficient for the purpose for which it is intended. The attempt to find the degree of reasonableness of the investment, which requires an impracticable amount of supervision, should be abandoned.

When an article discarded is replaced by another, the cost of the new article should be added, and the original cost of the old one should be subtracted from the investment. Changes in land values do not affect the investment as defined; therefore, it is only necessary, in order to keep an investment up to date, to see that when renewals are made the cost of the new parts less the original cost of the renewed parts is added to the investment. The actual amounts of new capital paid in for enlargements of old and the creation of new enterprises must also be added to the investment. Adequate supervision to secure honesty is required.

In the competitive enterprises which are used to determine their average profits, the same principles of ascertaining the investments and keeping their amount up to date must be followed. For this regulation, one valuation for every old enterprise is considered, and determinations of the investments in new enterprises are required. The author recommends what he calls the "unlimited life method" of providing for depreciation. The "actual life method" would more accurately describe it. It is a great improvement over the extremely

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artificial methods, not in agreement with actual depreciation, which are often advocated.

To some extent, however, his method is marred by his implied assumption, that the usefulness or serviceability of an article remains the same from the beginning to the end of its life as a part of the enterprise. Usually, this is not the case; his estimates of depreciation are thereby falsified. The consequent error is opposite to that resulting from his neglect of interest as a factor in depreciation, so that the two errors partly balance each other, and his estimates may thereby, by chance, sometimes become correct ones. In his view, the only cause of depreciation is the approaching obligation to renew, and the loss of usefulness and the increasing cost of maintenance of many parts of an enterprise are additional causes of depreciation. Maintenance may be defined as the replacement of parts, and renewal as the replacement of wholes. They can only be defined and distinguished by defining what is a part and what a whole. The various parts of every whole have different lives. In a new article few parts soon reach the end of their life; as the article grows old, more and more do so. The cost of maintenance of most articles, therefore, increases with age, and their net usefulness decreases to the same extent. For calculating the real net earnings of any enterprise, the determination of the real depreciation of its parts is important, and the author's discussion, though marred by errors, is, nevertheless, valuable, as it combats successfully and exposes many widespread errors and erroneous methods based thereon. The just and economical regulation of the natural monopolies under private ownership requires definite contracts, between the public and the owners, which secure efficient management without excessive supervision, by giving to the owners compensation in proportion to efficiency, and which make the returns on invested capital practically the same in competitive and monopolized industries. This requires prices varying with the material costs of production.

This regulation is practicable where the material causes of differences in cost of production and their effect on it can be estimated accurately. It can be introduced successfully, also, in semi-competitive industries like interstate railways. In these, part of the business is competitive, and part, mostly the local business, is not subject to competition. Competition, as modified by control, prescribes equality of rates for equal quality and quantity of service to the buyer by different competitors, and differences of rates for different services in proportion to the value of the service to the buyer; but it does not prescribe the absolute size of the rates. Just rates, which comply with all the requirements imposed by competition and vary with material costs of production, can be fixed. To comply with the requirements of competition it will be necessary to make the profits

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of each railway on competitive business what is obtainable, and to even up things by allowing different profits on non-competitive business. The rates for each road must be determined so that its total earnings are the same as would result from the rates proper if there were no competition.

In determining the local freight and passenger rates of old companies, average profits on their values, as previously determined, must be considered as a part of the costs of production of all the services rendered by them. For new railroads, which are built while the new system of rate regulation is in force, the proper local rates cannot be determined without knowing their value, and this cannot be found from their revenue; therefore, it must be assumed to be equal to the cost of production.

When the public allows an increase of earnings to the railroads of an economic district in proportion to the new investments, it is vitally interested to prevent unprofitable ones. Therefore it must forbid the building of unprofitable roads, or the adoption of uneconomical plans for those it allows, by requiring previous approval of all new railroads by the public authority and by watching the expenses. This is far different from accepting the cost of production of enterprises built without public supervision as their value, and allowing an average return on it.

The present ignorance regarding the principles which will govern the valuation and future rate regulation of interstate railroads is a great evil, increasing the risk of investment and consequently the cost of the capital needed for their further development. An authoritative utterance of these principles can be brought about by intelligent public discussion of these questions. This, therefore, is very desirable.

The author, by describing clearly and endeavoring to improve a bad method of monopoly control, shows up its defects and will thereby hasten its abandonment.

The foregoing criticisms are largely directed against this bad method, and not against the author's very able and lucid description of it. His paper clears up much confusion; it combats successfully many current errors, and suggests the correct solution of such problems as are not completely solved by him. The paper, therefore, is valuable, and well worth careful study; the author has rendered a great service by its presentation.

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THE WATER-PROOFING OF SOLID STEEL-FLOOR RAILROAD BRIDGES.

Discussion.*

By Messrs. Albert J. Himes, William S. Babcock, Jonathan Jones, J. Lee Allen, Glenn B. Woodruff, G. J. Ray, and J. B. W. Gardiner.

Mr. Himes.

Albert J. Himes,† M. Am. Soc. C. E. (by letter).—Mr. Wagner's paper is particularly useful, because the water-proofing of bridge floors has not yet crystallized into standard practice, and engineers are often in doubt as to the best design in a given case. The paper relates more particularly to the application of asphaltic compounds and the common fabrics used therewith.

It is worth while to state that under certain conditions a water-tight bridge floor can be constructed of concrete without water-proofing; that where concrete is not water-proof, it can be made so with the use of water-proofing; and that where both concrete and water-proofing are used, it frequently happens that the floor still leaks because neither the concrete nor the water-proofing is of good quality. It is desired to emphasize the need of first-class workmanship, and to warn engineers that the quality of workmanship necessary to secure water-proof bridge floors is exceedingly rare.

There are many reasons for the failure of attempts to construct water-tight bridge floors; one of the principal ones being that the duty required of the floor is too remote from the mind and observation of the workmen. If the work happens to be done in pleasant weather, it may be several months before any failure in the workmanship will be shown by the appearance of leakage on the under side

* This discussion (of the paper by Samuel Tobias Wagner, M. Am. Soc. C. E., published in December, 1914, *Proceedings*, and presented at the meeting of January 6th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cleveland, Ohio.

Mr. Himes.

of the floor, and the importance of unremitting care throughout the whole job is not impressed sufficiently on the workman's mind.

Another reason for the occasional failure of water-proofing is that the work is frequently timed so as to be completed during the wet, cold weather of the fall, when the conditions are most unfavorable for work of this class. Men who would do thoroughly satisfactory water-proofing in dry, pleasant weather, during the summer, will fail completely when, because of the cold and wet, it is difficult, inconvenient, and uncomfortable to clean and prepare properly the surfaces to be coated with water-proofing, and when the water-proofing materials are apt to be chilled during their application.

Another unfavorable condition is that under which it is necessary to maintain regular traffic. In such a case the necessities of transportation are always regarded as of greater importance, and there is a strong tendency to waive precautions and care because of the difficulty involved in caring for the running tracks while the water-proofing is in progress. Two failures of this character in the writer's experience will be mentioned. First, a concrete arch was to be built, and it was particularly desired that it should be water-proof. The matter was carefully presented to the foreman in charge of the work, who appeared to understand the situation thoroughly and, therefore, was relied on to make water-tight concrete, no water-proofing being used. Soon after the removal of the forms, various leaks were discovered, much to the chagrin of those in charge of the work.

At a later date, a similar arch was to be constructed under the same conditions, but, with the bad example for reference, the necessary care and attention were not wanting, and the second arch proved to be entirely satisfactory.

In another case, a double-track bridge floor, consisting of a concrete slab supported on floor-beams and stringers, was to be water-proofed while traffic was maintained on one track. The plan was to water-proof one-half of the bridge first, then to lay a track on that side, remove the track from the other side, and complete the water-proofing. In order to secure a tight joint on the axis of the bridge, the fabric was allowed to project loosely about 12 in. beyond the center line, thus providing sufficient lap.

The work was being done under the inspection of employes of another road who were disposed to be very critical and exacting. Instructions were given to the foreman to carry out their directions to the letter, and without any controversy. In the course of events, the inspector demanded that the loose flap of fabric which had been provided for the lap-joint should be cut off. The foreman complied with his directions, and the result was that, when the work was complete, a longitudinal seam existed along the axis of the bridge, which was anything but water-tight.

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In the design of a concrete floor-slab, it should always be remembered that cracks will be produced either by contraction or flexure, and joints should be provided with sufficient frequency and in such locations as to forestall the cracks.

A square street crossing, without curb supports and about 50 ft. in length, can be made water-proof with concrete alone. The introduction of curb supports will cause points of contraflexure over the supports, where cracks in the concrete are unavoidable. At such places, sheets of lead or copper may be inserted in the concrete with a fold to provide for a slight motion, and the floor can thus be made secure against the passage of water. If the bridge is on a skew, the design of such a joint becomes very difficult and sometimes impossible. Where water-proofing is used to prevent the water from passing through such cracks, the fabric is likely to tear, thus defeating its purpose.

The writer has endeavored to carry water from the bridge floor over the back-walls into wells built to take care of it, but agrees with Mr. Wagner that the best way is to carry it directly through the floor with down-spouts into the sewers. The down-spouts should be encircled with at least two flanges, which should be built into the concrete floor, not placed against the upper and lower surfaces. Flanges thus built into the concrete will effectually stop the passage of water along the pipe.

The writer does not agree with Mr. Wagner in his preference for a pocket of mastic along the web of the girder. It is better to rivet a flange angle along the web and to use malleable cast flanges around the stiffeners and gusset-plates. The water-proofing can then be carried up the side of the girder under the flange angle, and the joint will thus be sealed absolutely against the passage of water.

Turning to the specifications, comments are made as follows:

2.—Grades of 1% on the bridge floor are very difficult to secure, and may require an excessive weight of concrete. A thickness of concrete of 2½ in. is not sufficient to cover permanently the steel-work of the floor.

3.—The inlets should be provided with flanges, as previously described.

4.—Connecting longitudinal girders rigidly over a column causes motion in the bridge under the passage of trains. This motion in time develops looseness of joints, rust, and wear. Whenever it is desirable that girders should be thus connected, there should be such surplus of stiffness as to do away quite completely with the deflection.

10.—The protection of water-proofing with a layer of brick has been found in some cases to result in tearing the fabric of the water-

Mr. Himes.

proofing, because of the motion of the brick. This method, therefore, is not always satisfactory.

11.—Any pipe lying on the bridge floor is very likely to become clogged with cinders, and would not be recommended by the writer. He would prefer to allow the water to flow freely on the surface of the bridge and to provide as many down-spouts as possible.

13.—It would be better to use 1:2:4 concrete and do away with the ½-in. coating of cement mortar on the top. If the concrete is well made, the ½-in. coating is unnecessary, and it very often happens that the coating is placed so that it soon separates from the concrete below.

39.—Where water-proofing is to be used, a special effort should be made in planning the work to time its completion so that the water-proofing can be done during pleasant summer weather.

41.—The pocket of asphalt provided is inferior to the use of flange angles and malleable castings at the stiffeners.

44.—The writer would inquire whether any one has found a thin layer of sand on top of the water-proofing, covered by a thin layer of concrete to protect the latter against injury by picks and tamping bars, to be a better protection to the water-proofing than the layer of bricks described.

52.—This clause of the specification is probably intended to insure that the water-proofing will not be done in unfavorable weather.

54.—This clause is very general. A detailed description of the care to be exercised would be more useful.

In Appendices A and C, descriptions are given of certain tests to determine the quality of the asphalt. The writer would inquire whether the careful performance of these tests would insure a satisfactory quality of asphalt with the same certainty that one feels in the testing of cement or steel.

In Fig. 1, Appendix F, it is to be noted that the gutter projects below the bottom of the girder flanges and, therefore, diminishes the clearance beneath the bridge. This is a condition which is not permissible in city streets. The expense of securing the necessary clearance is too great.

It is a satisfaction to note the passing of the excessively shallow floor without ballast. The noise from these floors is intolerable. Henceforth, only ballasted bridges should be built, and the writer is inclined to think that the use of sand for ballast may be found more satisfactory than stone.

Mr. Babcock.

WILLIAM S. BABCOCK,* ASSOC. M. AM. SOC. C. E. (by letter).—The methods suggested by Mr. Wagner for the water-proofing of steel-floor railroad bridges are of the very best and most approved, but

* New York City.

the paper is misleading as to the materials generally used, inasmuch as Mr. Wagner makes no reference to coal-tar pitch or tar-saturated felt. Mr. Babcock.

Within the last 6 years, there have been approximately 1 000 bridges water-proofed with these materials, and in the majority of cases, good reports have been received. The method used has been practically the same as that described in the author's "General Specifications", but the materials have been straight-run coal-tar pitch and a 14 to 15-lb. coal-tar saturated felt, with a core of felt reinforced with cotton drilling.

For this type of water-proofing, the following materials have been used: 4 plies of 14-lb. tar-saturated felt; 1 ply of reinforced cotton drilling felt; and 250 lb., per 100 sq. ft., of straight-run coal-tar pitch.

The protection for the water-proofing which has been used was similar to that described under "Protection of Water-Proofing" by Mr. Wagner, with the exception that coal-tar pitch was used in the mastic, instead of asphalt. The preparation for water-proofing, using coal-tar pitch, would be the same as that suggested in the paper.

Referring to the statement, "The specifications, as presented, have been the outgrowth of about eight years' experience * * *," the following might be of interest:

The earliest data the writer has regarding the inception of bridge water-proofing were obtained on the New York Central and Hudson River Railroad. About 1888, that Company began the water-proofing of trough bridges on the Park Avenue Viaduct, in New York City. This water-proofing consisted of a binder, in some cases of coal-tar pitch, and in other cases of asphalt. This was not at all satisfactory. From this arose the necessity of producing flat-surface bridges of concrete.

Table 1 gives some tests of water-proofing asphalts. The author does not state when or where these tests were made, nor whether the analyses refer to asphalts which gave good, bad, or indifferent results, which leads one to inquire as to the usefulness of the results in connection with this paper.

Appendix E contains extracts from an article by Mr. J. W. Howard entitled "Asphalt Paving Cements and Road Binders", and the author states that, although written in reference to roads and pavements, it has much of interest in connection with the subject of asphalts for water-proofing purposes. He also states that these features may be elaborated by those interested in the subject, and ultimately form the basis of more specific data for water-proofing specifications.

Without definitely taking exception to Mr. Howard's statements, as quoted by the author, the writer desires to call to the attention of engineers the paper by Mr. A. W. Dow on "Relation Between Some

Mr. Babcock. Physical Properties of Bitumens and Oils."* In this paper, Mr. Dow sets forth some theories which would explain the apparent peculiarities of different types of bitumen, and his theories have not since been controverted successfully; in the writer's judgment, they may well form the basis of a study of the properties necessary in a bitumen for water-proofing purposes.

Mr. Jones. JONATHAN JONES,† ASSOC. M. AM. SOC. C. E. (by letter).—During 1914, the City of Philadelphia constructed a five-track bridge, of 90 ft. clear span, to carry the Philadelphia and Reading Railway tracks. The floor design and details were as shown in Fig. 9, and the specification for water-proofing was an earlier form of that proposed in this paper. In this earlier form, considerable latitude was given to the engineer, as to certain details, so that in effect the bridge may be said to have been water-proofed according to the specification presented in the paper.

The price paid to the sub-contractor for water-proofing was about 47 cents per sq. ft. of horizontal projection, including all construction above the steel floor-plate. This is a lower price than would usually be obtained.

The work was carefully followed up, and not a move was made except under the eye of the inspector; the design and specification, therefore, must take the blame if good results are not obtained in this case; at present they appear to be entirely satisfactory. The bridge has been under traffic for several months, and has been subjected to all kinds of weather conditions, including melting snow and ice, and it is perfectly water-tight.

In carrying out this work and observing its behavior, several points have occurred to the writer as needing further consideration before the present design and specification may be considered as standard.

It is not clear that the $\frac{1}{2}$-in. steel plate used on the floor-beams is superior to a non-reinforced concrete slab, haunched down on the beams and girders. Such a slab would preclude the necessity of painting the beams, and as its soffit can be kept parallel to the surface of the water-proofing, it will be a tougher body than the minimum of $2\frac{1}{2}$ in. shown in Fig. 9. It may be said that the steel plate is one additional water-proof membrane, but the plain slab will neither crack under traffic nor be pervious if properly poured, and it will be tighter along the girders, where trouble may be expected if it occurs at all. The saving in cost of the slab should pay for the extra metal in the girders due to increased dead load, and, if so, it is known that, of two bridges costing equal amounts, the heavier is, other things being equal, the better purchase.

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* *Proceedings*, Am. Soc. for Testing Materials, Vol. VI, p. 497.

† Philadelphia, Pa.

Mr. Jones.

It may prove necessary to insulate the 4-in. drain pipe and carry it into the abutment beyond the possibility of freezing. On sunny, winter days, the snow and ice on the bridge deck will melt and trickle down the drain, but as this is shaded and cold, the tricklings will freeze until the pipe is choked. Another rain following this occurrence, of course, will create a condition not cared for by the design.

There is considerable wasted dead load in the concrete haunches, and it would be well to consider burying vitrified sewer pipes, with end closures, in these haunches to save a little in first cost and to reduce the weight. There would still be ample bulk of concrete for the duty it performs.

Instead of caulking with asphalt, as specified in Paragraph 12, good results have been obtained by using cement mortar. These bridges are usually painted a light tint on the under side, to brighten the street, and when the asphalt used in caulking runs down through the joints, as it inevitably will, the appearance from below is very unsatisfactory.

Instead of the cement mortar coat specified in Paragraph 13, the writer advocates a granolithic mortar, as the former will always hair-crack on drying out. An expert sidewalk finisher is needed for this part of the work, as the 1% grade specified is only the minimum for carrying off the water, and no chance should be taken of losing any part of this grade by irregular finishing.

Incidentally, the last sentence of Paragraph 46 may be questioned as having been carried over from earlier designs and being inapplicable to those of Figs. 8 and 9. The writer believes it would be of value if Mr. Wagner would explain his preference for a protective covering partly of brick and partly of reinforced concrete, rather than wholly of one or the other; also his preference for bedding the brick in cement mortar rather than in asphalt. It would seem that the cement mortar will unite the brick rigidly to the felt, and when the bricks crack apart under vibration, the cracks will be communicated to the felt; but, with a yielding bond, such as asphalt, this would not occur.

All such water-proofing systems must be considered as tentative until they have had the test of years. The curse of all such systems is due partly to the possibility of deterioration within themselves, and partly because of the clogging of the ballast. Mr. Wagner, however, may be congratulated immediately on having gotten away from metal flashings as his main reliance. Those flashings were a torment to the drafting-room and bridge shop. The writer knows of one large contract on which the shop cost was actually doubled by the everlasting patching and fitting of those flashings around the stiffeners. One great advantage of the design proposed by Mr. Wagner is that fitting is done in the field with plastic materials and not with metal.

Mr. Allen.

J. Lee Allen,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has been for years interested in showing the permanency of the results obtained by the use of European asphaltic limestone rock mastic.

The author, after personal examination, shows some of the excellent results obtained from the use of European natural asphaltic rock mastic as a water-proofing medium, in which examinations the asphaltic rock mastic was found in splendid condition after 20 years' service. This perfection can be secured to-day, as the same materials are available, and, to make sure of their validity, the following clause should be added to the general specifications:

"The asphaltic rock mastic blocks shall be imported European asphaltic rock mastic, brought to the place of work in original blocks with the brand stamped thereon. The mastic blocks shall be shipped to the site of the work on a through bill of lading."

This clause would eliminate all the so-called mastics manufactured in the United States, which are made without the use of the natural European asphaltic limestone, and in which the manufacturers attempt to reproduce, in a few moments, what it has taken Nature ages to accomplish, and this is the reason for its failure. The great value of the European asphaltic rock mastic lies in the close relation between the bitumen and the mineral portion of the material, the former thoroughly impregnating the latter and resulting in a dense and stiff mass, although still very elastic and not susceptible to climatic changes, as a result of the soft condition and highly cementitious nature of the bitumen.

W. H. Finley, M. Am. Soc. C. E., refers to bridges† water-proofed in 1905 by the use of imported asphaltic rock mastic which have proved very satisfactory.

The writer, during his experience and observations of the result of the use of European asphaltic rock mastic for water-proofing purposes other than bridges, has found the materials perfect and still in daily service after 30 years of continuous use.

Mr. Woodruff.

Glenn B. Woodruff,‡ Jun. Am. Soc. C. E. (by letter).—There is probably no question more studied by the present-day railway bridge designer than that of water-proofing solid-floor bridges satisfactorily. At the same time, there is little published information relative to the experience of different engineers on this subject, and therefore the author deserves the thanks of those working on the problem for giving the results of his experiences. In some respects, however, it seems that there is room for a difference of opinion.

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* Chicago, Ill.

† "Waterproofing of Engineering Structures," *Journal*, Western Soc. of Engrs., Vol. XVII, p. 545.

‡ South Bethlehem, Pa.

Mr Woodruff.

One of the most important problems is that of drainage, which resolves itself into the proposition of removing the water from the bridge in the shortest possible time and in such a manner that there is the least chance of any moisture getting between the water-proofing and the steel. The writer believes that in structures of moderate spans the floor should be graded so as to force the water over the back wall, where it may be cared for by building a concrete gutter or even by a hollow tile drain lying on the sub-grade. If the attempt be made to carry the water through the bridge floor, it is a difficult matter to flash the down-spout so that no water will find its way under the water-proofing. Also, a screen small enough to hold back the smaller pieces of ballast is very likely to become clogged. Even where a means of cleaning this drain is provided, it is more than likely that it will not receive attention until the water makes its appearance at some undesired point.

The author's detail of substituting a pocket of asphalt mastic for the customary flash angle has advantages until the asphalt hardens or otherwise deteriorates. It may be questioned whether a flash angle above this asphalt pocket would not form sufficient additional protection to warrant its use.

The writer objects to placing the water-proofing membrane directly on the steelwork. In case this membrane grows hard and cracks, if it becomes damaged in any way, or if there is a slight air space between the steel and the membrane so that there is a possibility of condensation, water comes in contact with the steel under conditions favorable for corrosion and in a place where it cannot be detected. For this reason it is believed that a minimum of 3 or 4 in. of concrete, well reinforced, should be placed between the steel and the water-proofing. This concrete should be as dense as possible, and, in each case, experiments should be made with the aggregate to determine the proper mixture. The addition of a small percentage of hydrated lime is advisable.

Following out this scheme, the writer believes that a reinforced concrete slab carried by transverse rolled beams is, in many respects, at least, more satisfactory than the trough type of floor. In different cases, the writer has made comparisons of the cost of the two types and generally has decided in favor of the former. The same scheme may also be used to advantage in double-track bridges where the conditions demand a very shallow floor.

Mr. Ray.

G. J. Ray,* M. Am. Soc. C. E.—This paper is particularly valuable to those who have to do with the design and construction of solid steel-floor railway bridges. The author's exposition of "Preparation for Water-Proofing" and "Materials and Application" will be found

* Hoboken, N. J.

Mr. Ray.

especially valuable in the selection of a suitable water-proofing material, and its proper application, whether or not the structure to be water-proofed is similar in design to those described in Appendix F.

The speaker has found a material lack of reliable data and needed instructions for the design and application of water-proofing on various kinds of railroad bridges, outside of those recommended by the numerous firms selling such materials. In fact, one is apt to discredit much that is claimed for various water-proofing materials when one finds that nearly all water-proofing salesmen are inclined to run down the materials sold by their competitors.

The author's contribution, therefore, is especially valuable, as it outlines the subject from a practical viewpoint and the specifications are based on actual experience and results attained. Again, he has not confined the paper to one particular method of application.

The title of the paper, as well as the first paragraph of the synopsis, limits the consideration to "bridges with solid steel floors". Again, all the floor details given in Appendix F are of the steel-trough or steel-plate floor design. The speaker assumes, however, that it is in order to discuss the design and water-proofing of steel bridges of any other type with solid floor, which answers the same purpose as those described.

The first of the solid steel-floor bridges, with which the speaker had any experience, was built between 1903 and 1906. The floors of the deck bridges were made of channels, with the flanges up, and riveted together. The floors of the through bridges were of the ordinary trough type. With both through and deck bridges enough concrete was placed to provide good drainage. In most cases, the water was run off the bridges at one end, as they were on heavy grades. Plates were turned down over the back wall and, although fairly good drainage was provided, this construction did not prevent the water from finding its way through the back wall and down over the face of the abutment. No water-proofing was placed on the deck, and it was soon found that the ballast would have to be removed, and water-proofing applied. The water found its way down through numerous cracks formed in the concrete on top of the solid steel floor.

Five-ply felt, with brick protection, was used later when these structures were water-proofed. This water-proofing has eliminated the leakage through the floor system, but there is still some trouble with seepage through the back wall and over the abutment. Where this trouble was serious, the nuisance, to a great extent, has been eliminated by constructing a copper gutter along the abutment under the bridge seat.

In 1907 there was built, on the Delaware, Lackawanna and Western Railroad, a six-track deck-girder bridge, with a reinforced concrete floor 84 ft. wide and 349 ft. long. The bridge is at the neck of a

Mr. Ray.

yard with numerous cross-overs and slip switches on the bridge, so that the floor had to be sufficiently strong to permit the maximum loading at all points. A reinforced concrete deck, 1 ft. thick, was placed directly on top of the girders. Drainage outlets through this floor were provided at low points in each 814 sq. ft. of surface. As the bridge was rebuilt under traffic, the floor could be built for only two tracks at a time. A ridge in the concrete was made at the construction joints to prevent leakage, but no water-proofing was applied. Although this floor was built and put in service 7 years ago, it is in good condition to-day, and no leakage has occurred. Other bridges have since been built in the same manner and with equally good results. In one case, in 1908, the speaker built a deck-girder bridge, 91 ft. wide, with a clear span of 60 ft., over a city street, with a reinforced concrete floor, without the application of water-proofing. The floor is in perfect condition to-day, and has never leaked a drop; neither has any water found its way over the back wall to the bridge seat. The floor is 16 in. at the center and 13 in. at the ends, and the slab extends past the back wall, with a groove in the underside to intercept the seepage.

At first, the speaker had some doubt as to the success of the last-mentioned construction over city streets, as there is no question about the destructive tendency of water in passing through concrete. Therefore, nearly all the solid-floor bridges on his road have been water-proofed, regardless of the type of construction. During the past five years, his company has built deck-girder bridges over city streets with concrete slab floors, similar to those described in the paper; also numerous through-girder ballast-floor bridges with reinforced concrete slabs over the floor system, and also with the entire floor system—or bridges as a whole—encased in concrete. As a rule, the concrete for one or more tracks is poured complete in one operation, thus avoiding construction cracks at points where leakage is likely to occur. Where construction joints are necessary, copper flashing strips can be built into the concrete on each side of the joint, thus eliminating the leakage. These bridges, as a rule, have been water-proofed with five-ply felt, with brick protection, or with treated burlap and asbestos felt with asphalt mastic protection. There have been only a few leaks with either method, and up to date no trouble has been experienced from leakage through the sliding joint between the floor slab and the back wall. A double layer of treated felt or cotton drill has been placed in this joint with good results, and in all cases great care has been taken to extend the water-proofing at least 18 in. down over the back wall, and provide a good drainage system to carry off the water.

The only indication of water passing through the floor system has been a very slight seepage in a few places along the girders where the steel was painted prior to the construction of the concrete floor. The

Mr. Ray. speaker believes that the surest way to prevent this trouble is to eliminate painting of steelwork which is to be covered with concrete.

With long structures, the author's method of taking care of the drainage should produce good results where the drain pipes can be connected directly to the sewer, without traps, so that the pipes will not freeze during cold weather. With short bridges over city streets, it is difficult to maintain longitudinal drains under bridge floors, and down-spouts at abutments to street sewers. With a temperature of about 32° Fahr., ice and snow will melt on the bridge floor and freeze in the leaders and drains under the bridge. This difficulty is not so great where columns are placed on the curb line, and down-spouts can be carried directly to the sewer by vertical pipes. It will be found, however, that, in cold climates, such drains are constantly freezing, and are a source of expense to maintain.

Where columns are permissible on the curb line, the speaker has used steel construction encased in concrete for the long span over the paved portion of the street and reinforced concrete slabs over the sidewalks. This construction permits of a perfect job of water-proofing, and eliminates any possibility of water getting down the face of the abutment from seepage through the back wall. In such cases, the columns have been constructed of concrete; the depth required from the top of the rail to the under-clearance does not exceed 4 ft., 7 in., for spans up to 50 ft. centers. This type of construction, as a whole, is not unreasonably expensive. The encased steelwork is costly, but the slabs over the sidewalks are economical, on account of the short spans, and the thickness of the abutments can be reduced materially. Standard track centers can be maintained by keeping the center girder shallow. Painting is entirely eliminated, and the design can be made ornate enough to please the most skeptical critic.

On eight through-girder bridges recently constructed, the cost of the concrete floor, including reinforcement and encasement of I-beams, averaged 35.6 cents per sq. ft. of floor space. These bridges varied in length, from center to center of bearings, from 53 ft. 8 in., to 120 ft. 7 in., and the depth from top of rail to under-clearance of girders varied from 3 ft. 8 in., to 4 ft. All water-proofing was done with three-ply treated burlap and asbestos felt, protected with asphalt mastic, 1½ in. thick. The average cost of water-proofing was 28 cents per sq. ft., thus making the entire cost of the floor system, 63.6 cents per sq. ft., exclusive of the structural steel. The steel structures were no heavier than would have been required by the method shown by Fig. 9.

In conclusion, it is the speaker's opinion that:

1st.—The specifications for materials and method of applying them, as set forth by the author, should obtain good results.

2d.—The specifications for details of construction for the types of structures described in Appendix F, in general, should insure workmanlike finish and secure good results, especially where the drainage system can be connected directly to the sewers without intervening traps. Mr. Ray.

3d.—With through-girder construction, and a depth of from 3 ft. 8 in., to 4 ft., from top of rail to under-clearance, a floor system can be designed which will permit of encasement in concrete, thus eliminating painting, and insuring a water-tight job at less expense than the cost of the floor shown by Fig. 9.

4th.—With deck-plate girder bridges, a water-tight reinforced concrete floor, without water-proofing, can be constructed which will not exceed 12 in. in thickness.

5th.—Where water-proofing is to be applied to any type of solid-floor bridge, the kind of materials to be used should be determined before the details of construction are fixed.

J. B. W. GARDINER,* ESQ.—Before making any comment on this paper, the speaker wishes to express both to Mr. Wagner and to the Society his appreciation of their courtesy in granting him the privilege of participating in the discussion of a most interesting subject. Mr. Gardiner.

With a keen and thorough knowledge of the conditions to which a water-proofing on the floor of a steel bridge is subjected, Mr. Wagner has given a very complete specification which the asphalt should fulfill. Not that any asphalt which will fill this specification will necessarily prove an efficient water-proofing material; but probably it will. However, if it cannot fill this specification, it most certainly will not prove efficient. It is thought, however, that it should be a part of the specification that the asphalt should be unaffected by the alkalies in cement mortar or the acid-impregnated drippings that come through cinders, and that it should adhere strongly to the concrete and to the membrane in conjunction with which it is used.

The question of a proper water-proofing membrane, however, is deserving of more extended discussion than Mr. Wagner has given it. Although it is unquestionably true that no membrane is of itself a water-proofing agent, the function which it performs in a water-proofing system is fully as important as that performed by the plying cement. They are entirely interdependent. The membrane is a binder—and a binder only—and as such its function is to hold in place the asphalt which is expected to shed the water and carry it to the drains. In the course of application, the membrane and the plying cement are, or should be, firmly cemented together so that they are practically one. Consequently, any movement of one part of the system occurs simultaneously in the other. Therefore, in so far as they

* New York City.

Mr. Gardiner.

are applicable, the specifications for the membrane should demand the same qualities as those for the plying cement. To illustrate: Mr. Wagner's asphalt specification requires ductility through a wide range of temperatures, which means that it must remain firm and elastic under extreme changes of weather. This quality in asphalt is essential, first, that it may not be shattered by the shock of impact when a moving train first passes on the bridge, or by the vibration of the entire structure due to the moving load; and secondly, the asphalt must yield or stretch to meet both the slight deflection which occurs as the center of mass shifts with the load and the movement in the concrete base incident to temperature changes. Expansion joints, it is true, eventually take care of this movement, but they do not do so immediately. By this, the speaker means that the movement is taking place throughout the entire length of the slab and is merely summed up at the expansion joints. That is, the movement which occurs at the expansion joint is a summation of an indefinite number of smaller movements which take place between the center of the slab and the expansion joint. If then we encase the ductile asphalt between binders of a rigid inelastic membrane, we have sacrificed all the advantages derived from this ductility. To be logical, therefore, we should insist that the elasticity of the membrane be equal to the ductility of the asphalt.

Another point to be considered is the presence of oils which show more than a certain percentage of volatility when subjected to a temperature of 325° Fahr. for a specified time. Far fetched as it may seem, this test has a distinct bearing on membranes. In order that the membrane may have the maximum protection against rotting, it should be thoroughly saturated with a stable asphalt. Under no circumstances should bitumen solvents be used in this process. In many cases, water-proofing membranes are treated by immersing them in a bath of asphalt liquefied by the addition of petroleum residue, in the proportion of 1 part asphalt to 4 or 5 parts residue. This applies principally to felts and burlaps. The result is saturation, it is true, but saturation largely with the petroleum residue, which, being a bitumen solvent, will eventually re-act against the plying cement. The specifications, therefore, should require that the membrane be saturated with an asphalt which would comply with Mr. Wagner's specification. As Mr. Wagner has pointed out, flashing effectively against the web of a half-through girder is an extremely difficult problem, largely because of projecting stiffeners, gussets, knee braces, etc. This difficulty is not lessened by the use of a stiff non-flexible membrane which cannot be made to fit snugly against the web at all points. Flexibility would seem, therefore, to be an important quality of the membrane. This

fact is largely responsible for the use, first, of burlap, and, later, of cotton fabric as water-proofing membranes. Mr. Gardiner.

A few words as to the relative merits of cotton, and of burlap, which is a jute fiber, might not be misplaced. Jute consists of the chemical compound of cellulose with lignin, to which some investigators have given the name, bastose. It is much more readily affected by the action of acids and alkalies than cotton fiber, which is 95% pure cellulose. The influence of moisture, and even of air, will also rot the jute fiber. These characteristics go far toward fixing the cause of the trouble that so many engineers have experienced with water-proofing of this type.

Two other points in connection with membranes deserve passing mention, namely, resistance to puncture, and tensile strength. The desirable resistance to puncture, measured in pounds per square inch, by a standard paper-testing machine, should be determined by the chief engineer. The minimum tensile strength should be sufficient to stand the test for elasticity, without fracture, with a reasonable factor of safety.

As to the method of application; there is one point mentioned by Mr. Wagner, the reason for which the speaker does not understand, and in which, so far as his knowledge of the subject goes, he cannot agree. That is, the requirement that the first layer of felt should not be cemented to the floor of the bridge. If there is merit in it, it would seem logical to lay the protecting brick course in a sand cushion instead of cement mortar, so as to keep the water-proofing course separate and distinct from either the base or the protection. On the other hand, one of the cardinal principles of good application is the avoidance of bridges or spans, so that when weight is placed on the water-proofing it will not break because of lack of support. This can only be obviated by making the first layer of membrane conform absolutely to the surface; and this is possible only by cementing the first layer to the surface thoroughly at all points. The possibility of puncture from other causes is also much more remote if this method is used.

The method described by Mr. Wagner of finishing the water-proofing against the girders or concrete is decidedly the most satisfactory of any in the speaker's experience. However, he can see no other object in painting the surface with which the sealing material comes in contact before putting such material in place, than to secure satisfactory cohesion. As there are materials on the market, sold under the name of expansion joint cements, which cohere perfectly without a primer, it would seem cheaper and safer to use them.

As to methods of protecting a water-proofing course on a solid-floor bridge, the speaker knows of no more satisfactory method than brick laid in cement grout, being careful to fill all joints in the brick by

Mr. Gardiner.

pouring mortar over its surface after it has been laid, and sweeping it thoroughly into all joints. It does little good to fill the joints of the brick with asphalt, as even the best hard burnt bricks are porous and offer little obstruction to the passage of water. The use of an asphalt mastic for a protecting coat is an expensive luxury, and by no means a necessity. The argument, at best, for indulging in this luxury is that it is an additional safeguard against the failure of the water-proofing. If the water-proofing is efficient, there is no need of this additional precaution, and it is useless to put in any water-proofing material unless it is good.

One more point is that we are living in an age of intense specialization, and it is possible, to-day, to obtain specialized service without paying a premium. Therefore, the method of letting a water-proofing contract, under which a water-proofing manufacturer is required to do all the concrete work, as well as applying the water-proofing and putting in place the protection coat, does not work out to the best advantage of the railroad. The water-proofer can lay the water-proofing material and place the protection. That is his business, in which he has specialized. He is not a specialist in concreting. The railroad would get better and cheaper concrete, either by letting that work to a contractor whose business is concrete, or by doing it itself. This plan has the disadvantage of being a little more troublesome, and, to a certain extent, of decentralizing responsibility. On every piece of construction work, however, railroads always maintain an inspector to see that the specifications are carried out, so that these objections are of little moment. As far as is known, it is the general custom on all but four of the Eastern roads, to let a general contract for all the work to a concrete man and to require him to have the water-proofing done by experienced water-proofers with a material approved by the chief engineer. Of these four, two do all their own concreting and water-proofing with their own crew, purchasing the material they prefer to use. The other two use the method advocated by Mr. Wagner.

As to water-proofing specifications generally, it does not seem as though we had progressed as rapidly in their formulation as in specifications for other forms of structural work. Our engineers are thoroughly aware of conditions which must be met. They are familiar, too, with the general processes of manufacture of the various materials which are designed to meet these conditions. It would not then seem a difficult task to draw a standard specification which would demand the qualities which a water-proofing course should have, and require the manufacturers to produce such a product. Such a move would spur the manufacturer toward improvement, standardize the water-proofing industry, and give the railroads improved service and ever-improving materials.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ROBERT MAXSON GREENE, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1914.

Robert Maxson Greene, the son of David Maxson and Emma MacAlpine Greene, was born in Hillsdale, N. Y., on March 30th, 1870, and received his early education in the public schools of that place and elsewhere.

Prior to entering Rensselaer Polytechnic Institute, where he studied engineering for three years, until June, 1892, he had been employed, from July, 1887, to January, 1889, as Roadmaster's Assistant in the Engineering Department of the Missouri Pacific Railway, at Pueblo, Colo., and Scott City, Kans.

Mr. Greene had always been interested in structural and architectural work, and in September, 1892, he was appointed a Draftsman in the Structural Department of the Pencoyd Iron Works, Philadelphia, Pa., where he remained until January, 1894. From January to May, 1894, he was employed as Architect by the New Jersey Building and Supply Company, and from May to December, 1894, as Draftsman and Checker by the Jackson Architectural Iron Works of New York City.

In December, 1894, Mr. Greene went to Pittsburgh, Pa., as Draftsman with the Pittsburgh Bridge Company. He remained with this Company until July, 1895, when he returned to New York City and was engaged with Milliken Brothers as Draftsman and Assistant Engineer until November, 1898. During this time his work consisted of detailing and checking the structural steel and iron work for office and mill buildings, elevated railways, bridges, viaducts, etc., and, among other work, he had charge of the structural details for the following: Museum of Arts and Sciences and Polhemus Memorial Clinic Buildings, Brooklyn, N. Y.; the tower of St. Luke's Hospital and repairs to the United States Appraisers Warehouses, in New York City; six-story department store, in Scranton, Pa.; Leary Building, Seattle, Wash.; Terminal Station of the Chicago and North Western Railway, Chicago, Ill.; Hearst Building, San Francisco, Cal.; Lafayette Hotel, Buffalo, N. Y.; fifteen-story railway building in Buenos Aires, Argentine Republic; sugar factory buildings in the Philippine Islands; etc., etc.

From November, 1898, to January, 1899, Mr. Greene was employed as Designing Engineer by De Lemos and Cordes, Architects, of New York City. In January, 1899, he became a member of the firm of

* Memoir prepared by George E. Thackray, M. Am. Soc. C. E.

Foster and Greene, Architectural Engineers, of New York City, and on the dissolution of the partnership, in October, 1900, the firm was continued as Greene and Ward, Engineers and Contractors. These firms had charge of designing, detailing, and furnishing the structural steel and iron work for the Pugh Building, the Stewart Apartment House, and various other apartment houses and private houses, public schools, the First Battery Armory, Holy Trinity Church, etc., in New York City; the Roman Catholic Church, at Corona, Long Island; the Winter Club House, at Tuxedo Park, N. Y., etc., etc.

In November, 1901, Mr. Greene retired from the firm of Greene and Ward to engage in private practice, in New York City, as a Designing and Consulting Engineer. In this capacity, he had charge of the detailing, purchase and erection of the structural and ornamental steel and iron work for various buildings for the Sisters of St. Joseph at Brentwood, Long Island, a barracks for the United States Marine Corps, at Washington, D. C., and many other structures.

In September, 1903, Mr. Greene accepted a position with the Cambria Steel Company, of Johnstown, Pa., as Designer and Estimator on structural steel and iron work for bridges, buildings, viaducts, tunnels, etc.; he remained in this position until October, 1906. From December, 1906, until 1912, he was in the employ of the American Bridge Company of Ambridge, Pa., as Assistant Engineer in charge of detail construction of steel and iron work for office and mill buildings, hotels, etc.

From 1912 until his death on December 5th, 1914, Mr. Greene was engaged in private practice as a Structural Engineer, first in Detroit, Mich., and later in Winnipeg, Man., Canada. He had been in ill-health for the last few years of his life, but died suddenly while in Chicago, Ill., from hemorrhage and heart failure. He was married, on November 18th, 1896, at Manhasset, Long Island, to Miss Harriette Simpson Horsfield, who survives him.

He was a most pleasant gentleman, always thoughtful of the welfare of his companions and associates, and his early and sudden death will be regretted by his numerous friends throughout the United States.

Mr. Greene was elected an Associate Member of the American Society of Civil Engineers on June 1st, 1898, and a Member on September 1st, 1908.

HENRY LEWIS OESTREICH, M. Am. Soc. C. E.*

Died August 13th, 1914.

Henry Lewis Oestreich was born in New York City, on February 13th, 1870. He was of German parentage, his father coming from Eckelsheim-on-Rhine, and his mother from Heidelberg. He

* Memoir prepared by F. C. Noble, M. Am. Soc. C. E.

received his early education in the public schools of New York City, and learned so rapidly that he completed his preparatory studies before he was old enough to enter college. At this period of his life, he was uncertain for what calling to fit himself. His own idea was to become a physician, but his mother wished him to be a civil engineer, and lived long enough to see him receive the degree of B. S. C. E. from New York University, in 1888. He was the youngest graduate in his class.

Mr. Oestreich's first engineering experience was gained during his last college vacation, as Chainman and Topographer on railway construction in Virginia. After his graduation, he was employed for about a year at general city engineering work in the office of Mr. B. Hufnagel, in Mt. Vernon, N. Y. In July, 1889, he entered the office of the late Charles B. Brush, M. Am. Soc. C. E., who lectured at New York University and knew of Mr. Oestreich's abilities as a student. From this time until April, 1897, he was engaged as Assistant Engineer in charge of various engineering works in and near Jersey City, including the construction of the Weehawken Viaduct, Weehawken Reservoir No. 2, Hillside Road, and part of the Hudson County Boulevard, including bridges and sewers.

In April, 1897, Mr. Oestreich entered the service of New York City as Assistant Engineer in the Department of Highways, where he remained in responsible charge of highway construction until May, 1900. He then secured a transfer to the New York Rapid Transit subway work, at that time just beginning under William Barclay Parsons, M. Am. Soc. C. E. He was engaged on this work until his death, 14 years later. Here his capacities were recognized by promotion, with the result that, during most of this period, his position was that of Resident Engineer in charge of important and complicated subway contracts; he stood at the head of a large corps of assistant engineers and inspectors.

Mr. Oestreich's qualities of mind and character were greatly admired by those who knew him. He was an indefatigable worker, painstakingly attentive to details, but never losing perspective on that account. His ready grasp and fair, common-sense treatment of an engineering situation or matter in controversy could always be depended on. His ideals of duty and truthfulness were uncommonly high, and that they governed his every action was a fact to be appreciated in proportion to one's acquaintance with him. Added to these qualities, his sincerity, even temper, modesty, consideration, and courtesy endeared him to his friends and were recognized by all whom he met. It is certain that his progress in the Engineering Profession would have been continuous and rapid if it had not been for his untimely end.

Mr. Oestreich was married on June 10th, 1914, to Miss Stella B. Eccles, of Trenton, N. J., and his death was especially sad, as it occurred so soon after his marriage.

Mr. Oestreich was elected a Junior of the American Society of Civil Engineers on October 4th, 1892; an Associate Member on December 6th, 1899; and a Member on April 6th, 1909.

LUCIAN ARNOLD TAYLOR, M. Am. Soc. C. E.*

DIED NOVEMBER 19TH, 1914.

Lucian Arnold Taylor, the youngest son of Jared and Catherine (Truesdell) Taylor, was born at Harrisville, R. I., on June 20th, 1846. Until he was sixteen years of age, his education was confined to winter schooling, the remainder of the year being spent on his father's farm at Woodstock, Conn.

At the outbreak of the Civil War, Mr. Taylor tried to enlist but, being only fifteen years old, he was not allowed to do so until July 15th, 1862, when he enlisted in Company B, 18th Connecticut Volunteers, and served until the close of the war, in 1865. He was in active service all the time except for about four months, when he was confined in the Southern prison at Richmond and at home recovering from that experience. At the close of the war, he returned to Woodstock and spent the next two years working on the farm and attending Woodstock Academy.

In 1867, Mr. Taylor moved to Worcester, Mass., and attended Howe's Business College. In 1868, he entered the employ of the City of Worcester, as an Assistant to the City Engineer, Mr. Phinehas Ball. He remained in the City Engineer's office until 1882, having direct charge of several important pieces of work, chief among which were the rebuilding of the Leicester Dam after its failure in 1876; the construction of the Island sewer, involving heavy and difficult work, in 1878 and 1879; and the building of the first Holden Dam, in 1880.

In 1882, Mr. Taylor was elected Water Commissioner, continuing in that office until 1885 when he resigned to enter the contracting firm of William C. McClallan and Company, of Boston.

After his partner's death, in 1886, Mr. Taylor bought Mrs. McClallan's interest in the firm, and from that time until his death carried on the business as a Consulting Engineer and Contractor. The water and sewerage systems of many New England cities and towns were built by him, among the most important being those of

* Memoir prepared by C. M. Allen, M. Am. Soc. C. E.

Bar Harbor, Kingfield, Wells, and Cumberland, Me.; Newport, N. H.; St. Albans and St. Johnsbury, Vt.; Lynn, Orange, Quincy, Webster, Millbury, Concord, Provincetown, and Rutland, Mass.; and New Haven and Groton, Conn.

The fact that many of Mr. Taylor's contracts included both the engineering and the construction indicates the high esteem which his clients had for his judgment and integrity. Often he was invited to bid without competition or with only a limited number of bidders, and so satisfactory were his services that, in later years, many of his engagements were in communities where he had formerly been employed.

During the later years of his life, he did less contracting and more consulting work. When forced by illness to give up his practice in 1913, he was Consulting Engineer for the City of Lynn, and the Town of Falmouth, Mass., and had been President of the Millbury, Mass., Water Company for several years.

Although Mr. Taylor's office was in Boston for the last twenty-nine years of his life, he retained his residence in Worcester, where his death occurred on November 19th, 1914, after an illness of more than a year, having been caused by a complication of diseases.

He was married on August 29th, 1868, to Jeannette Arnold, of Putnam, Conn., who, with one son and one daughter, survives him.

He was a member of the Boston Society of Civil Engineers, the Engineers Club of Boston, the American Water-Works Association, the New England Water-Works Association, the Worcester County Society of Engineers, the Worcester County Mechanics Association, and the George H. Ward Post, G. A. R.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on November 4th, 1891.

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"Nomographic Solutions for Formulas of Various Types." R. C. STRACHAN. (To be presented Feb. 3d, 1915.).....Dec., 1914
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"RIVERS AND RAILROADS IN THE UNITED STATES." WILLIAM W. HARTS. (To be presented Feb. 3d, 1915.)

"RECONSTRUCTION OF THE NORFOLK AND WESTERN RAILWAY COMPANY'S BRIDGE OVER THE OHIO RIVER AT KENOVA, WEST VIRGINIA." WILLIAM G. GROVE and HENRY TAYLOR. (To be presented Feb. 17th, 1915.)

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"Reinforced Concrete Docks: Foreign and American Structures. Failures, Costs, and General Considerations." HARRISON S. TAFT.....Apr., "
Discussion. (Author's Closure.).....Aug., Sept., Dec., 1914, Jan., 1915
"Some Principles Relating to the Administration of Streams." CLARENCE T. JOHNSTON.....May, 1914
Discussion.....Sept., Oct., Nov., Dec., "
"The Construction of the Klondike Pipe Line." W. W. EDWARDS.....May, "
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"External Corrosion of Cast-Iron Pipe." MARSHALL R. PUGH.....Aug., 1914
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"Investigation of the Performance of a Reaction Turbine." R. L. DAUGHERTY..Oct., 1914
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"Penstock and Surge-Tank Problems." MINTON M. WARREN.....Oct., "
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"Submerged Pipe Work at Portland, Oregon." D. D. CLARKE.....Nov., 1914
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"History of Little Rock Junction Railway Bridge of the St. Louis, Iron Mountain and Southern Railway Company, Over the Arkansas River, at Little Rock, Arkansas, 1883-1914." C. E. SMITH.....Nov., 1914
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"Proof of an Assumption in the Theory of Concrete Beams." RALPH E. GOODWIN.....Nov., 1914
"Depreciation as an Element for Consideration in the Appraisal of Public Service Properties." C. E. GRUNSKY.....Nov., "
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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XLI—No. 2

FEBRUARY, 1915

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NEW YORK 1915

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TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, Frank M. Kerr.

ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

ON FLOODS AND FLOOD PREVENTION: C. McD. Townsend, John A. Bensel, T. G. Dabney, C. E. Grunsky, Frank M. Kerr, Morris Knowles, J. B. Lippincott, Daniel W. Mead, John A. Ockerson, Arthur T. Safford, Charles Saville, F. L. Sellew.

TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Robert W. Hunt, George W. Kittredge, Paul M. LaBach, C. G. E. Larsson, William McNab, G. J. Ray, Albert F. Reichmann, F. E. Turneaure, J. E. Willoughby.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

SIXTY-SECOND ANNUAL MEETING*

January 20th, 1915.—The meeting was called to order at 10 A. M.; President Hunter McDonald in the chair; Charles Warren Hunt, Secretary; and present, also, about 600 members.

The Secretary presented the report of the Tellers appointed previously by the Board of Direction to canvass the Ballot for Officers for the ensuing year.

The President announced the election of the following officers:

President, to serve one year:

CHARLES D. MARX, Stanford University, Cal

Vice-Presidents, to serve two years:

DANIEL BONTECOU, Kansas City, Mo.

CLEMENS HERSCHEL, New York City

* A full report of the Sixty-second Annual Meeting is printed on pages 71 to 91 of this number of *Proceedings*.

Treasurer, to serve one year:

LINCOLN BUSH, New York City

Directors, to serve three years:

JOHN V. DAVIES, New York City
GEORGE A. HARWOOD, New York City
J. E. GREINER, Baltimore, Md.
JOHN F. COLEMAN, New Orleans, La.
JOHN B. HAWLEY, Fort Worth, Tex.
HERBERT S. CROCKER, Denver, Colo.

The Annual Report of the Board of Direction, and the Annual Reports of the Secretary and of the Treasurer,* for the year ending December 31st, 1914, were presented and accepted.

The Secretary read the report of the Committee appointed by the Board of Direction to recommend the award of prizes,† and announced that the Medals and Prizes for the year ending July, 1914, had been awarded by the Board of Direction in conformity with that report, as follows:

THE NORMAN MEDAL to Paper No. 1260, entitled "Hydrology of the Panama Canal", by Caleb Mills Saville, M. Am. Soc. C. E.

THE J. JAMES R. CROES MEDAL to Paper No. 1254, entitled "Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct", by J. B. Lippincott, M. Am. Soc. C. E.

THE THOMAS FITCH ROWLAND PRIZE to Paper No. 1270, entitled "Irrigation and River Control in the Colorado River Delta", by H. T. Cory, M. Am. Soc. C. E.

THE JAMES LAURIE PRIZE to Paper No. 1275, entitled "The Elevation of the Tracks of the Philadelphia Germantown and Norristown Railroad, Philadelphia, Pa.", by Samuel Tobias Wagner, M. Am. Soc. C. E.

THE COLLINGWOOD PRIZE FOR JUNIORS to Paper No. 1264, entitled "Experiments on Weir Discharge", by W. G. Steward, Esq., and J. S. Longwell,‡ Jun. Am. Soc. C. E.

The Secretary presented the report of the Tellers appointed by the Board of Direction to canvass the suggestions as to members of the Nominating Committee, and the following were appointed to serve two years:

CHARLES HANSEL.....*Representing District No. 1*
JAMES W. ROLLINS, JR..... " " " *2*
J. A. O'CONNOR..... " " " *3*

* For these reports, see pages 21 to 30 of *Proceedings* for January, 1915 (Vol. XLI).

† See page 73.

‡ The Collingwood Prize for Juniors was awarded to Mr. Longwell for his participation in the preparation of this paper, Mr. Steward not being connected with the Society.

W. C. Cushing.....*Representing District No. 4*
Alexander Maitland, Jr.... " " " 5
P. H. Norcross..... " " " 6
Charles Derleth, Jr..... " " " 7

The consideration of the following proposed amendment to the Constitution was then taken up:

Amend Article VII — Nomination and Election of Officers — as follows:

Strike out Section 1, and substitute the following:

"The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year."

Strike out the first paragraph of Section 2, and substitute the following:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, shall be appointed by the meeting to serve for two years. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve Districts; and these, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society."

Strike out the word "and" in the fifteenth line of the third paragraph of Section 2, and after the figure "7" add: "8, 9, 10, 11, 12, and 13."

On motion, duly seconded, the amendment was amended by changing parts of the fifth and sixth lines of the paragraph relating to Section 2 to read as follows: "and these, with the last five living Past-Presidents of the Society, shall be", etc.

W. W. Crosby, M. Am. Soc. C. E., Chairman of the Special Committee on Materials for Road Construction and Standards for Their Test and Use, presented a Progress Report of that Committee.*

The Committee was continued.

A. L. Bowman, M. Am. Soc. C. E., Chairman of the Special Committee on Steel Columns and Struts, presented a Progress Report of that Committee.†

On motion, duly seconded, it was ordered that the report be printed in *Transactions*.

* Printed on pages 2997 to 3050 of Papers and Discussions in the *Proceedings* for December, 1914.

† Printed on pages 3051 to 3062 of Papers and Discussions in the *Proceedings* for December, 1914.

Nelson P. Lewis, M. Am. Soc. C. E., Chairman of the Special Committee to Investigate the Conditions of Employment of, and Compensation of, Civil Engineers, presented a Progress Report of that Committee.*

On motion, duly seconded, the Committee was continued.

Richard L. Humphrey, M. Am. Soc. C. E., Secretary of the Special Committee on Concrete and Reinforced Concrete, stated that the Committee had nothing to report at this time.

This was accepted as a Progress Report.

Frederic P. Stearns, Past-President, Am. Soc. C. E., Chairman of the Special Committee on Valuation of Public Utilities, presented a Progress Report of that Committee.†

The report was accepted and the Committee continued.

Robert A. Cummings, M. Am. Soc. C. E., Chairman of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, presented a Progress Report of that Committee.‡

On motion, duly seconded, the report was accepted and the Committee continued.

The Secretary read a communication from Desmond FitzGerald, Past-President, Am. Soc. C. E., Chairman of the Special Committee on Engineering Education.

The Committee was continued.

F. H. Newell, M. Am. Soc. C. E., Chairman of the Special Committee on A National Water Law, reported progress.

The Committee was continued.

C. McD. Townsend, M. Am. Soc. C. E., Chairman of the Special Committee on Floods and Flood Prevention, reported progress.

The Committee was continued.

A. N. Talbot, M. Am. Soc. C. E., Chairman of the Special Committee to Report on Stresses in Railroad Track, presented a Progress Report of that Committee.§

The Committee was continued.

The Secretary read the Rules adopted by the Board of Direction for the Appointment, Government, and Procedure of Special Committees, January 6th, 1915.‖

The Secretary announced the establishment of a fund, termed the Engineering Foundation, which is to be "devoted to the advancement of the Engineering Arts and Sciences in all their branches, to the greatest good of the Engineering Profession, and to the benefit of Mankind." This fund is to be administered by a board in which the American Society of Civil Engineers, the American Institute of Mining Engi-

* Printed on pages 3063 to 3071 of Papers and Discussions in the *Proceedings* for December, 1914.

† See p. 82.

‡ See p. 491 of Papers and Discussions in this number.

§ See p. 85.

‖ Printed on pages 7 to 9 of *Proceedings*, for January, 1915.

neers, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers are to be equally represented. The inaugural meeting is to be held at 8.30 P. M. on Wednesday, January 27th, 1915, in the Auditorium of the Engineering Societies Building, 29 West 39th Street, New York City.

The Secretary announced that the next Annual Convention of the Society would be held in San Francisco, Cal., on September 16th to 18th, 1915, these days immediately preceding the opening of the International Engineering Congress, on September 20th, 1915.

John A. Ockerson, Past-President, Am. Soc. C. E., offered the following resolution:

"*Whereas:* Many distinguished members of the American Society of Civil Engineers, including six Past-Presidents, have been prominently identified officially with the several Canal Commissions and are largely responsible for the location, development of plans, and inauguration of construction work, all of which have resulted in a successful Isthmian Canal, and

"*Whereas:* The Chief Engineer of the Canal and Governor of the Canal Zone and his chief associates, whose energy and ability have carried to a successful completion the greatest of all engineering works, are also members of this Society, therefore, be it

"*Resolved:* That this Society should be formally represented at the opening ceremonies of the Panama Canal by a suitable delegation of its members, and that official sanction and recognition for such participation be requested from the proper authorities in charge thereof; and be it further

"*Resolved:* That the Board of Direction be and is hereby requested to take such steps as may be found necessary to carry these resolutions into effect."

On motion, duly seconded, the resolution was passed.

Charles D. Marx, President-elect, Am. Soc. C. E., addressed the meeting on the subject of the International Engineering Congress.

Mr. Charles C. Schneider and Mr. Charles Macdonald conducted Mr. Marx to the chair.

Mr. Marx addressed the meeting briefly.

Adjourned.

SPECIAL MEETINGS FOR THE DISCUSSION OF THE PROGRESS REPORT OF THE SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION

January 22d, 1915. — The meeting was called to order at 10.15 A. M.; President Charles D. Marx in the chair; Charles Warren Hunt, Secretary; and present, also, 175 members and guests.

The President announced that the special meetings were called to afford an opportunity for members of the Society and non-members to discuss the 1915 Report of the Special Committee on "Materials for Road Construction and Standards for Their Test and Use". The President then requested W. W. Crosby, M. Am. Soc. C. E., to serve

as Chairman of the meetings and A. H. Blanchard, M. Am. Soc. C. E., to serve as Secretary.

Written discussions submitted by Messrs. C. S. Reeve, John S. Miller, Jr., and Clifford Richardson were read by the Secretary. The Report was then discussed by Messrs. James Owen, E. W. Stern, T. Hugh Boorman, W. H. Fulweiler, J. E. Myers, W. H. Kershaw, Philip P. Sharples, R. A. Meeker, W. M. Kinney, Hunter McDonald, Theron M. Ripley, Edwin H. Thomes, and Robert A. MacGregor.

Adjourned.

January 22d, 1915.—The meeting was called to order at 2.15 P. M.; W. W. Crosby in the chair; A. H. Blanchard, acting as Secretary; and present, also, 145 members and guests.

The discussion of the 1915 Report of the Special Committee on Materials for Road Construction was continued by Messrs. Theron M. Ripley, Bertrand H. Wait, E. M. Vail, Edwin H. Thomes, W. B. Spencer, T. Hugh Boorman, George P. Hemstreet, R. A. Meeker, P. A. Dallis, Philip P. Sharples, L. L. Tribus, Leonard C. Jordan, H. W. Durham, Robert A. MacGregor, W. T. Hedley, William H. Connell, and C. R. Allen, Jr.

Adjourned.

REGULAR MEETING

February 3d, 1915.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Charles Warren Hunt, Secretary; and present, also, 90 members and 17 guests.

The minutes of the meetings of December 16th, 1914, and January 6th, 1915, were approved as printed in *Proceedings* for January, 1915.

A paper by R. C. Strachan, M. Am. Soc. C. E., entitled "Nomographic Solutions for Formulas of Various Types", was presented by the author and illustrated with lantern slides. The paper was discussed by Messrs. J. O. Eckersley and R. H. Merrill, and communications on the subject from Messrs. Allen Hazen and Carl B. Andrews were read by the Secretary. Other communications on the subject, from Messrs. Charles W. Schubert and Paul C. Nugent, on account of their mathematical nature, were read by title only.

A paper by William W. Harts, M. Am. Soc. C. E., entitled "Rivers and Railroads in the United States", was presented by the Secretary who also read a communication on the subject from Walter S. Wheeler, M. Am. Soc. C. E.

The paper was discussed by F. Lavis, M. Am. Soc. C. E.

The Secretary announced the following deaths:

John Wykeham Jacomb-Hood, of London, England, elected Member February 5th, 1902; died in March, 1914.

Charles Ezekiel Rasinsky, of Cincinnati, Ohio, elected Associate Member July 9th, 1912; died December 26th, 1914.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

January 20th, 1915.—The Board met during the Annual Meeting, as required by the Constitution, at the House of the Society, January 20th, 1915, at 2.10 P. M., President Marx in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bensel, Bontecou, Bush, Coleman, Connor, Crocker, Davies, Edwards, Endicott, Fuller, Greiner, Harwood, Haskell, Hawley, Herschel, Hodge, Keefer, Leonard, Loweth, McDonald, Metcalf, Montfort, Ockerson, Swain, Tuttle, and Williams.

The following Standing Committees were appointed:

Finance Committee: Henry W. Hodge, George W. Fuller, Clemens Herschel, Leonard Metcalf, and John F. Coleman.

Publication Committee: James H. Edwards, J. E. Greiner, Arthur S. Tuttle, Henry R. Leonard, and Gardner S. Williams.

Library Committee: George A. Harwood, J. V. Davies, H. S. Crocker, M. E. Cooley, and Chas. Warren Hunt.

The President announced that the first business was the election of a Secretary.

Mr. Hunt retired.

Chas. Warren Hunt was nominated and the nomination was seconded.

Past-President Mordecai T. Endicott was nominated and the nomination was seconded.

Mr. Endicott retired.

A ballot was taken, which resulted in the election of Mr. Hunt.

Mr. Endicott was recalled.

Mr. Hunt was recalled, and the President announced to him his election.

It was decided that the next meeting of the Board be held on Tuesday, March 2d, 1915, and that subsequent regular meetings of the Board be held bi-monthly on the Tuesday before the first Wednesday in May, July, September, November, and January.

The resolution offered by Past-President Ockerson at the Annual Meeting relating to the representation of this Society at the opening ceremonies of the Panama Canal, which was referred to the Board of Direction by the Annual Meeting for action, was considered, and the President was authorized to appoint a Committee of three, of which he shall be Chairman, with power to carry out the purport of the above resolution.

The President subsequently appointed to act with him on this Committee Messrs. Endicott and Ockerson.

The following resolution was adopted:

"Resolved: That, in the opinion of the Board of Direction of the American Society of Civil Engineers, it would be unfortunate for

the present Alaskan Railway Commission to be superseded, and that the interest of the Public demands that the present Commission be allowed to carry out the construction as well as the location of the line."

The President was requested to write to the President of the United States calling his attention to this resolution.

Adjourned.

REPORT IN FULL OF THE SIXTY-SECOND ANNUAL MEETING, JANUARY 20TH AND 21ST, 1915

Wednesday, January 20th, 1915 (10 A. M.)—Hunter McDonald, President, in the chair; Charles Warren Hunt, Secretary; and present, also, about 600 members.

Meeting called to order.

THE PRESIDENT.—Gentlemen, please come to order.

Our first business will be the report of the Tellers on the ballot for officers. The Secretary will please read the report of the Tellers.

Report of Tellers on Ballot for Officers.

THE SECRETARY.—The Tellers appointed to canvass the ballot for officers of the Society report as follows:

"220 West 57th Street,
New York, N. Y.

"JANUARY 20TH, 1915.

"TO THE SIXTY-SECOND ANNUAL MEETING
"AMERICAN SOCIETY OF CIVIL ENGINEERS:

"The Tellers appointed to canvass the ballots for Officers of the Society for 1915 report as follows:

"Total number of ballots received		3 387
Ballots without signatures	13	
" with illegible signatures	0	
" stamped, not signed	4	
" from members in arrears of dues	18	
Total number not entitled to vote		35
Ballots canvassed		3 352
Defective		48

"For President:

CHARLES D. MARX	3 274
Scattering	11

"For Vice-Presidents:

DANIEL BONTECOU	3 240
CLEMENS HERSCHEL	3 241
Scattering	17

"For Treasurer:

LINCOLN BUSH	3 278
Scattering	3

"For Directors:

"District No. 1.	JOHN V. DAVIES	3 013
	FREDERIC MOLITOR	1 188
	GEORGE A. HARWOOD	2 192
	Scattering	15

Report of Tellers (continued).

"District No. 4.	J. E. GREINER	3 241
	Scattering	9
"District No. 6.	JOHN F. COLEMAN	3 217
	JOHN B. HAWLEY	3 220
	Scattering	27
"District No. 7.	HERBERT S. CROCKER	3 240
	Scattering	9

"JOHN G. VAN HORNE,
"OTIS E. HOVEY,
"WILLIAM J. BOUCHER,
"WESTON E. FULLER,
"F. LAVIS,
"HOWARD B. BAIRD,
"D. L. TURNER,
"J. H. GRANBERY,
"FREDERICK C. NOBLE,
"ALBERT A. NORTHROP,
"Tellers."

Officers Declared Elected.

THE PRESIDENT.—I now declare the following Officers elected as the result of this ballot: Mr. Charles D. Marx, President; Messrs. Daniel Bontecou and Clemens Herschel, Vice-Presidents; Mr. Lincoln Bush, Treasurer; Messrs. John V. Davies and George A. Harwood, Directors from District No. 1; Mr. J. E. Greiner, Director from District No. 4; Messrs. John F. Coleman and John B. Hawley, Directors from District No. 6; and Mr. Herbert S. Crocker, Director from District No. 7.

Report of the Board of Direction.

Our next business will be the Annual Report of the Board of Direction, which the Secretary will please read.

The Secretary presented the Report of the Board of Direction.*

THE PRESIDENT.—We will now hear the Report of the Secretary.

Report of the Secretary.

The Secretary presented his Report† of receipts and disbursements for the year.

THE PRESIDENT.—We will hear the Report of the Treasurer, if Mr. Wallace is here; if not, we will ask the Secretary to read it.

Report of the Treasurer.

The Secretary read the Report of the Treasurer.‡

THE PRESIDENT.—These Reports are before you, gentlemen. Unless some member wishes to discuss them we will consider them adopted. The Chair hears no discussion.

The next business is the award of medals and prizes for 1914.

Award of Medals and Prizes.

THE SECRETARY.—Mr. President, I will read the Report of the Committee appointed to recommend the award of prizes, dated December 29th, addressed to the Board of Direction.

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* See *Proceedings*, Am. Soc. C. E., Vol. XLI, p. 21 (January, 1915).
† See *Proceedings*, Am. Soc. C. E., Vol. XLI, p. 28 (January, 1915).
‡ See *Proceedings*, Am. Soc. C. E., Vol. XLI, p. 30 (January, 1915).

"*Board of Direction,*
American Society of Civil Engineers,
220 West 57th Street,
Borough of Manhattan, N. Y. City.

"DEC. 29TH, 1914.

"GENTLEMEN:—Your Committee, which was appointed to recommend the award of prizes for the year 1914 for papers submitted to and accepted by the Society, has carefully read and considered the merits of all the papers which were published and which are eligible under the established rules for competition for any one of the five prizes awarded at this time.

"In making the following recommendations we do not wish to imply that in our opinions no other papers are of sufficient merit to deserve such recognition. On the contrary, there are a number of very meritorious papers between which and those selected there are no great differences so far as merit is concerned and whose authors deserve commendation for the thorough and interesting manner in which they have treated their subjects. We also consider it a matter for regret that the excellent paper presented by Mr. W. E. Lilly, upon 'The Strength of Columns' is not eligible for this competition because its author is not a member of the Society.

"It is quite possible that others may not concur in our conclusions or that your Board may not approve of the recommendations but our decision has been reached after careful study and discussion and it is unanimous.

"We, therefore, respectfully recommend,

"1. That the Norman Medal be awarded to Mr. Caleb Mills Saville, for his paper upon the 'Hydrology of the Panama Canal';

"2. That the J. J. R. Croes Medal be awarded to Mr. J. B. Lippincott, for his paper upon 'Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct';

"3. That the Thomas Fitch Rowland Prize be awarded to Mr. H. T. Cory, for his paper upon 'Irrigation and River Control in the Colorado River Delta';

"4. That the James Laurie Prize be awarded to Mr. Samuel Tobias Wagner, for his paper upon 'The Elevation of the Tracks of the Philadelphia, Germantown and Norristown Railroad, Philadelphia, Pa.';

"5. That the Collingwood Prize for Juniors be awarded to Mr. J. S. Longwell for his participation in the preparation of the paper upon 'Experiments on Weir Discharge'.

"Mr. W. G. Steward who collaborated with Mr. Longwell in the preparation of this paper is not a member of the Society in any grade and, therefore, cannot participate in the award.

"Respectfully submitted,

"E. J. FORT,
"E. G. HOPSON,
"H. D. WILLIAR,
"Committee."

Mr. President, I have to report that the Board of Direction, on receipt of this report, unanimously awarded the prizes in accordance with the recommendations made.

THE PRESIDENT.—No further action is necessary on that report. The next business is the report on members to serve on the Nominating Committee for the next two years. The Secretary will read the result of the ballot on Members of the Nominating Committee.

Appointment of Nominating Committee.

THE SECRETARY.—The following is the Report of the Tellers appointed by the Board of Direction to canvass the suggestions for members of the Nominating Committee to be appointed by the Annual Meeting:

"District No. 1: Total suggestions received, 415, as follows:

Charles Hansel	208
C. W. Bryan	148
D. L. Turner	51
Scattering	8"

Mr. President, will you take up each section as it is read?

THE PRESIDENT.—We will take up one section at a time. The question of the appointment of the member of the Nominating Committee for District No. 1 is before you.

A. L. BOWMAN, M. AM. SOC. C. E.—I move that Mr. Hansel, having received the largest number of votes, be appointed a member of the Nominating Committee for District No. 1.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Charles Hansel be appointed a member of the Nominating Committee for District No. 1. All in favor of the motion will say "aye"; all opposed, "no". The motion is carried.

THE SECRETARY.—"District No. 2: Total suggestions received, 154, as follows:

James W. Rollins, Jr.	117
William H. Moore	27
G. E. Verrill	3
Scattering	7"

J. WALDO SMITH, M. AM. SOC. C. E.—I move that Mr. Rollins be appointed a member of the Nominating Committee for District No. 2.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. James W. Rollins, Jr., be appointed a member of the Nominating Committee for District No. 2. All in favor of the motion say "aye"; contrary, "no". Carried.

THE SECRETARY.—"District No. 3: Total suggestions received, 224, as follows:

J. A. O'Connor	176
E. A. Fisher	31
Walter McCulloh	8
Scattering	9"

THE PRESIDENT.—What shall we do with the report for District No. 3?

MR. J. WALDO SMITH.—I move that Mr. O'Connor be appointed a member of the Nominating Committee for District No. 3, he having received the largest number of votes.

THE PRESIDENT.—It is moved and seconded that Mr. J. A. O'Connor be appointed a member for District No. 3. All in favor of the motion say "aye"; all opposed, "no". Carried.

THE SECRETARY.—"District No. 4: Total suggestions received, 212, as follows:

W. C. Cushing	123
A. J. Himes	48
H. H. Quimby	34
Scattering	7"

ROBERT A. CUMMINGS, M. AM. SOC. C. E.—I move that Mr. Cushing be appointed as the representative on the Nominating Committee for District No. 4.

THE PRESIDENT.—It is moved and seconded that Mr. W. C. Cushing be appointed a member of the Nominating Committee for District No. 4; all in favor of the motion will say "aye"; all opposed "no". The motion is carried.

THE SECRETARY.—"District No. 5: Total suggestions received, 181, as follows:

Alexander Maitland	99
T. L. Condron	41
J. W. Alvord	22
Scattering	19"

J. A. OCKERSON, PAST-PRESIDENT, AM. SOC. C. E.—Mr. President, I move that Mr. Alexander Maitland be appointed as the representative of the Nominating Committee from District No. 5.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Alexander Maitland be appointed a member of the Nominating Committee for District No. 5. All in favor of that motion will say "aye"; all opposed, "no". The motion is carried.

Nominating Committee (continued).

THE SECRETARY.—"District No. 6: Total suggestions received, 142, as follows:

P. H. Norcross	73
J. E. Crawford	31
B. M. Hall	18
Scattering	20"

A MEMBER.—I move that Mr. Norcross be appointed a member of the Nominating Committee from the Sixth District.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. P. H. Norcross be appointed a member of the Nominating Committee for District No. 6. All in favor of the motion say "aye"; all opposed, "no". Mr. Norcross is appointed.

THE SECRETARY.—"District No. 7: Total suggestions received, 224, as follows:

Charles Derleth, Jr.	110
J. B. Lippincott	37
D. C. Henny	28
Scattering	49"

ALLEN HAZEN, M. AM. SOC. C. E.—Mr. President, I move that Mr. Charles Derleth, Jr., having received the largest number of suggestions, be appointed a member of the Nominating Committee from the Seventh District.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Charles Derleth, Jr., be appointed a member of the Nominating Committee from District No. 7. The motion is carried.

THE SECRETARY.—I have only to say that this report is signed by the tellers: Henry W. Hodge, Arthur S. Tuttle, and J. H. Edwards.

Proposed Amendment to the Constitution.

THE PRESIDENT.—The next business before the meeting is the consideration of the proposed Amendment to the Constitution. The Secretary will please read the Amendment.

THE SECRETARY.—The following proposed Amendment to the Constitution of the American Society of Civil Engineers, having been received by the Secretary prior to the first Wednesday in November, 1914, is herewith presented. It was sent by letter to all Corporate Members of the Society, and is in order for discussion at this meeting in accordance with Article IX, Section 2, of the Constitution.

"Amend Article VII—Nomination and Election of Officers—as follows:

"Strike out Section 1, and substitute the following:

"The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical dis-

tricts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year.

"Strike out the first paragraph of Section 2, and substitute the following:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, shall be appointed by the meeting to serve for two years. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve Districts; and these, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society.

"Strike out the word 'and' in the fifteenth line of the third paragraph of Section 2, and after the figure '7' add: '8, 9, 10, 11, 12, and 13'."

This Amendment is signed by the following corporate members: R. Montfort, W. A. Cattell, E. H. Connor, E. E. Haskell, and S. H. Hedges.

The President.—This proposed Amendment, gentlemen, is now before you for discussion, before it is issued to letter-ballot.

R. Montfort, M. Am. Soc. C. E.—Mr. President, I move that it be sent to letter-ballot, without further amendment.

Gardner S. Williams, M. Am. Soc. C. E.—There is one change in the language of that Amendment that I think would have to be made, as it is not proper English at present. I think it says "the five living last Past-Presidents." I beg to say that there is but one last Past-President, whether he is dead or any other way. I will offer an amendment to that to strike out the words "five" and "last", and let it stand with the "living Past-Presidents". I see no good reason why the Past-President representation on this Committee should be limited to five. I think it will be better for the Society if all living Past-Presidents are permitted to participate in the work of the Nominating Committee. They are able to take a better perspective view of the needs of the Society than any other men in it, and I think the Society should have the advantage of their knowledge, experience, and judgment in picking out those who are to help them as their officers thereafter.

I have not discussed this subject with anybody so far, and I do not know whether there is any one in the room who wishes to support this motion, but I make it in the hope that some one will hold the same view that I do.

A Member.—I would like to second that motion.

Discussion on Proposed Amendment to the Constitution.

THE PRESIDENT.—The motion is before you, gentlemen, for discussion.

A. D. FLINN, M. AM. SOC. C. E.—Mr. President, I would like to inquire whether Mr. Williams will accept the additional words, "Past-Presidents who continue to be members of the Society". I suppose it is improbable that any will drop out, but——

THE PRESIDENT.—Mr. Williams accepts that amendment to his proposed amendment. Gentlemen, Mr. Williams' amendment is before you for discussion. I think the first thing to do is to pass upon the suggestion that the language of the amendment be corrected, and I would ask Mr. Williams to state with precision exactly how he intended it to read.

MR. WILLIAMS.—If this amendment is carried, of course it eliminates that phase of the subject. If this amendment is voted down, then I will make a specific amendment to cover that.

THE PRESIDENT.—We will then consider first the motion to include all the living Past-Presidents as members of the Nominating Committee, and the question is before this meeting for discussion. I would be glad to hear from any members who wish to discuss it.

A. W. CARPENTER, M. AM. SOC. C. E.—I would like to ask how many Past-Presidents there are living?

THE PRESIDENT.—The Secretary reports eight.

THE SECRETARY.—Eight in addition to the five.

J. G. VAN HORNE, M. AM. SOC. C. E.—Would mileage have to be paid to the thirteen men?

THE SECRETARY.—Mr. President, I do not wish to be considered in opposition to any motion at all. I merely wish to call attention to the fact that it is a dangerous matter to adopt, off-hand, an amendment to the Constitution of the Society. We frequently find that one part of the Constitution does not quite agree with the others; and there should be a careful study made before any amendment is adopted. I find one clause here which may or may not have some bearing on this matter: "At all meetings of the committee [referring to the Nominating Committee] eight members shall constitute a quorum". I remember very well when the discussion on that matter took place in the Society at Seattle, and it was based very largely on the number of members of the Committee who were not Past-Presidents, there being seven districts and fourteen members appointed from those districts; and eight is a majority of those fourteen. I merely make that as a suggestion which should receive consideration.

THE PRESIDENT.—The Chair would call attention to the fact that this is a very radical change in our past practice, and the amendment should not be made unless it is thoroughly discussed.

GEORGE M. PURVER, ASSOC. M. AM. SOC. C. E.—I want to say, in reference to this amendment proposed, that we have before us now an

increase of districts, which makes from those districts fourteen members, and with the five Past-Presidents, gives us nineteen. I have no objection to all the Past-Presidents taking part in the nomination of the officers of the Society, but it is going to be unwieldy; I am afraid we are going to get too many members on the Nominating Committee, so that we will not get as good service as with a smaller number.

F. P. Stearns, Past-President, Am. Soc. C. E.—As one of the Past-Presidents, I wish to say a word. It seems to me that five Past-Presidents furnish a balance wheel, and then again, on personal grounds, I think that after a President has served for five years, in that capacity, after going out of office, he is entitled to a rest.

The President.—Is there any further discussion on Mr. Williams' motion? Are you ready for the question? All in favor of the adoption of Mr. Williams' amendment to the Constitutional amendment will say "aye"; all opposed, "no". The amendment is lost.

Mr. Williams now has another amendment, and if he will permit the Chair the suggestion, I think the verbiage can be made correct by striking out the word "last" and placing the word "latest" in front of the word "living", leaving it the "five latest living Past-Presidents". We will ask Mr. Williams then to fix that in the way it ought to be.

The Secretary.—Just a transposition of the word "last" from in front of "Past-Presidents" before the word "five", so that it will read "with the last five living Past-Presidents".

Mr. Williams.—That is correct. I will so move.

Motion seconded.

The President.—Is there any discussion? If not, all in favor of the motion will say "aye"; all opposed, "no". The motion is carried.

The question is now on the motion of Mr. Montfort to send the proposed amendment out to letter-ballot. Any discussion on the motion?

Mr. Hazen.—Mr. President, I believe most thoroughly in the division of the districts as proposed. From experience on the committee that struggled with this matter last year, I also appreciate the force of what the Secretary has said, and it will be embarrassing to adopt any amendment that might be in conflict with any other part of the Constitution; and, without delaying the matter, is it not possible to refer it to a committee to make sure that it is in proper form; or can we not get some assurance that it is in proper form as it stands, before letting it go forward?

The President.—I will ask the Secretary to reply to Mr. Hazen's inquiry.

The Secretary.—Mr. President, this Amendment to the Constitution was gotten up by a committee of the Board of Direction after studying the matter for several months; two years ago it came up

Proposed Amendment to the Constitution (continued).

at the Annual Convention at Ottawa and was amended, and on the ballot was defeated, because it had some other matter added to it.

The Board of Direction then took the matter up, appointed another Committee, and that Committee studied it over most carefully in relation to all the parts of the Constitution, and reported it in the same words as the previous Committee had done; so I think really this amendment has received very careful consideration, as it is now, with reference to other parts of the Constitution.

Mr. Hazen.—On the Secretary's assurance that this amendment is in proper form, I move that the amendment be submitted to the membership for vote.

Motion seconded.

The President.—Is there any further discussion on the motion? If not, all in favor will say "aye"; all opposed, "no". The motion is carried. The amendment will be sent out to letter-ballot.

Report of Committee on Materials for Road Construction.

We will now hear the Report of the Special Committee on Materials for Road Construction and on Standards for Their Test and Use, by Mr. W. W. Crosby, Chairman.

W. W. Crosby, M. Am. Soc. C. E.—Mr. President, the report is long, and has been printed. In its printed form it is much more intelligible than it would be on being read, and to save the time of the meeting, I would like to suggest that the report be received and read by title only at this time.

The President.—The suggestion of the Chairman is before you. Has Mr. Crosby's motion a second?

Motion duly seconded.

The President.—It is moved and seconded that the Progress Report of the Committee be received. All in favor of the motion will say "aye"; all opposed, "no".

The motion is carried.

Mr. Crosby.—Mr. President, may I make an announcement? Gentlemen, the printed report of this Committee is available, and, through the courtesy of the Board of Direction, Special Meetings of the Society have been arranged for Friday, of this week, at 10 a. m. and 2 p. m., at which time discussion of this report is invited. The Committee hopes that there will be a good attendance at those meetings, and that constructive criticism of the report will be presented to the fullest possible extent. The Committee would like the assistance which may be secured in that way.

Report of Committee on Steel Columns and Struts.

The President.—We will hear the Progress Report of the Special Committee on Steel Columns and Struts, Mr. A. L. Bowman, Chairman.

A. L. Bowman, M. Am. Soc. C. E.—Mr. President and Gentlemen of the Society, the Progress Report of your Committee has been

printed* in *Proceedings*. The tests mentioned therein are being continued.

At the meeting of the Board of Direction on June 2d, 1914, Mr. Rudolph P. Miller was appointed to fill the place of the late Emil Gerber; and I would like to say that the Committee appreciated Mr. Gerber highly, both the personality of the man and the work he did for the Committee.

I move that this Progress Report be printed in the *Transactions* of the Society, in order that this record of tests be preserved to our members.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that this Progress Report be printed in the *Transactions* of the Society. All in favor of the motion say "aye"; all opposed, "no". Carried.

W. W. CREHORE, M. AM. SOC. C. E.—(Mr. Crehore made a brief discussion, which will be found among the Papers and Discussions of this number of *Proceedings*.)

L. D. RIGHTS, M. AM. SOC. C. E.—(Mr. Rights, Secretary of the Committee, replied briefly to Mr. Crehore.)

ALEXANDER HARING, M. AM. SOC. C. E.—Mr. President, I move that this Progress Report be printed in the *Transactions* of the Society, so that this record of tests may be preserved for the use of the members.

Motion seconded.

THE PRESIDENT.—Gentlemen, you have heard the motion. All in favor say "aye"; all opposed, "no". The motion is carried.

Report of Committee on Employment and Compensation of Civil Engineers.

The next report is that of the Special Committee to Investigate the Conditions of Employment of, and Compensation of, Civil Engineers, Mr. Nelson P. Lewis, Chairman.

NELSON P. LEWIS, M. AM. SOC. C. E.—Mr. President and Gentlemen, the report has been printed in the *Proceedings* of the Society.† Is it your pleasure that I read it, Mr. President?

THE PRESIDENT.—No, I think we had better dispense with the reading of it, but we would be glad to hear whatever explanatory remarks you care to make.

(Mr. Lewis made some remarks explanatory of the Report.)

MR. WILLIAMS.—I move that the Committee be continued and instructed to use its discretion as to what steps it shall take to further this investigation.

Motion seconded.

THE PRESIDENT.—The motion is before you, gentlemen. Is there any discussion? All in favor of the motion will say "aye"; all opposed, "no".

* See *Proceedings*, Am. Soc. C. E., for December, 1914, pages 3051 to 3062 of Papers and Discussions.

† See *Proceedings*, Am. Soc. C. E., for December, 1914, pages 3063 to 3071 of Papers and Discussions.

The motion is carried.

Report of Committee on Concrete and Reinforced Concrete.

We will now hear from the Special Committee on Concrete and Reinforced Concrete, Mr. Joseph R. Worcester, Chairman.

Richard L. Humphrey, M. Am. Soc. C. E.—Mr. President, Mr. Worcester, I think, is not here. As the Secretary of the Committee, I would call the attention of the Society to the fact that at the last Annual Meeting a resolution was passed that the Committee would not be expected to report until it had something to report. The Committee has nothing to report at this time.

The President.—We will accept the Report of the Secretary, Mr. Humphrey, as a Progress Report.

The Special Committee on Engineering Education, Mr. Desmond FitzGerald, Chairman. Is Mr. FitzGerald present? We will pass that for the present. The Secretary has a communication from the Chairman, but he is out of the hall at present. We will take that up later.

Report of Committee on Valuation.

The report of the Special Committee on Valuation of Public Utilities, Mr. Frederic P. Stearns.

(Mr. Stearns presented the following statement of the work of his Committee):

"Your Special Committee, appointed to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, has no formal report to present at this meeting, but has requested its Chairman to make a brief statement of the progress made during the past year.

"It may be well to recall that a formal progress report of the Committee was presented at the last Annual Meeting of the Society, and that in accordance with a resolution offered by the Chairman at that meeting it was referred back to the Committee for further consideration.

"A special meeting for the discussion of the report was held March 11, 1914, and it was again discussed at an adjourned meeting held on the afternoon and evening of April 2, 1914. Written discussions were received for several months, the last being published in the *Proceedings* for August, 1914.

"In April the Committee suffered a great loss by the death of two of its valued members, Thomas H. Johnson and Alfred Noble, and a further vacancy was created by the resignation of Henry M. Byllesby. These vacancies were filled by the Board of Direction in May, by the appointment of Charles S. Churchill, William J. Wilgus, and Henry E. Riggs.

"The present Committee has held two meetings of three days each, one in October and one in December, with the full membership present at each meeting, and has carried on a voluminous written discussion. It has made an outline of a report, of which some chapters have been drafted, discussed, and tentatively agreed to. The writing of drafts of additional chapters has been assigned to different members of the Committee, and progress has been made on these drafts, but, as already

indicated, the work has not advanced far enough to permit the making of a formal report to the Society at this meeting.

"For the Committee:
"FREDERIC P. STEARNS,
"Chairman."

THE PRESIDENT.—Gentlemen, the report of the Committee is before you. Unless there is objection, it will be accepted as a Progress Report.

Report of Committee on Soils.

The Report of the Special Committee to Codify Present Practice on Bearing Value of Soils for Foundations, Mr. Robert A. Cummings, Chairman.

ROBERT A. CUMMINGS, M. AM. SOC. C. E.—Mr. President and Members of the American Society: This report consists of about 40 pages of typewritten matter. It is not my intention to read this to you, as it will appear in the *Proceedings* at a later date.*

(Mr. Cummings stated some of the main features of the Report.)

THE PRESIDENT.—Gentlemen, the report of the Committee is before you. The Chair awaits a motion as to its disposition. I feel quite sure that——

T. C. ATWOOD, M. AM. SOC. C. E.—Mr. President, I move that this Report be accepted, and that the Committee be continued. I think the Society is very much indebted to this Committee for the enthusiastic manner in which it has tackled this difficult and rather unintelligible subject, and should be allowed to go ahead in the same way.

Motion seconded.

THE PRESIDENT.—Gentlemen, you have heard the motion. All in favor say "aye"; all opposed "no".

Motion carried.

Report of Committee on Engineering Education.

We now revert to the Progress Report of the Special Committee on Engineering Education. The Secretary has a communication from the Chairman, which we will ask him to read.

The Secretary read the following letter:

"18th Jan. 1915.

"MY DEAR MR. HUNT.—As it seems highly probable that I may not be able to attend the Annual Meeting, will you say for me in regard to the Com. on 'Engineering Education' that the Carnegie Foundation has taken the matter in hand and is now making a careful investigation. The only action that our Committee can now take is to aid the Foundation in every way in our power.

"Very faithfully yours,
"DESMOND FITZGERALD,
"Chairman."

THE PRESIDENT.—Is there any action the Society desires to take with reference to this communication? If not, we will consider that the Committee has reported progress and will be continued.

* This Report is printed in this number of *Proceedings*.

Report of Committee on A National Water Law.

The Special Committee on A National Water Law, Mr. F. H. Newell, Chairman.

F. H. NEWELL, M. AM. SOC. C. E.—Mr. President and Gentlemen, I wish to make a very brief statement of progress of the Special Committee on A National Water Law. This Committee has been so well distributed geographically and is so widely apart in its individual views on the subject, that so far it has not been able to get together physically or mentally on the subject.

Moreover, the subject which has been assigned to it is of such far-reaching importance, going away back into the establishment of the Constitution of the United States, and from there on up into the latest freak of State legislation, that we have begun to appreciate that we have a very large task before us, and yet one which is of very great interest, and, as Chairman of the Committee, I hope that the Society will continue it, and that we will perhaps be constituted in such a way that we can get together and organize the Committee, perhaps, into sections, taking up in detail some of these very important points.

I may state, as showing progress, that although the Committee has not been able to get together, we have had a great deal of correspondence among the members, and we have sent out circulars—or the Secretary of the Society has sent out circulars—to all the membership, the replies from which indicate, as I have stated, very wide interest in the subject and very far-reaching importance, in that the substance of those replies indicates that investments aggregating many millions of dollars are being delayed because of uncertainties which grow out of a lack of adequate State and National water laws, and that the fullest use and enjoyment of our National resources are rendered imperfect through existing conditions which might be remedied through well-conceived legislation.

The suggestion has been made that the Committee may take up at the present time the line of investigation which has been entered upon, and endeavor to agree, first on the principles which would be embodied in any new legislation, if it is possible to elucidate this legislation, and to present them as the consensus of the opinion of the Society. It will then be practicable to endeavor to embody them in various drafts of the law.

Various other engineering and local organizations are interested in the subject, and when agreement can be had in part, at least on the broad principles, it is believed that a conference may be had with other organizations, in order that the subject may be presented from other standpoints, and thus secure wider agreements also on fundamentals.

I will not endeavor to discuss further this broad problem, but request, as stated before, that the Committee be continued in its work.

THE PRESIDENT.—Unless there is a discussion of this report of Mr. Newell, the Committee will be considered as continued.

Report of Committee on Floods and Flood Prevention.

The Special Committee on Floods and Flood Prevention, Col. C. McD. Townsend, Chairman.

C. McD. TOWNSEND, M. AM. SOC. C. E.—Mr. Chairman and Gentlemen, the Special Committee on Floods and Flood Prevention has held three meetings, or rather, attempted to hold three meetings. The membership—as it is necessary to get a good representation of the country—is scattered from Maine to California, but we have appointed a sub-committee to draft a report, and that committee has formed a rough draft, but has been unable to get a consensus of opinion of the membership of the Committee due to its being scattered over the United States, in time for this meeting, and I shall again have to ask an extension of time for this Committee, and can merely report progress.

THE PRESIDENT.—You have heard the report of the Committee. Unless there is a discussion or objection, the Committee will be considered as continued and its present report as one of progress.

The Report of the Special Committee on Stresses in Railroad Track, Professor A. N. Talbot, Chairman.

Report of Committee on Stresses in Railroad Track.

A. N. TALBOT, M. AM. SOC. C. E.—Mr. President, and Gentlemen of the Society, the Special Committee to Report on Stresses in Railroad Track presents the following report of progress:

"The Committee was formed by action of the Board of Direction, November 12th, 1913. A similar committee was created by the Board of Direction of the American Railway Engineering Association, November 20th, 1913. By action of these two bodies the two Committees have the authority to co-operate, and this co-operation is being carried on. The Board of Direction of the American Society of Civil Engineers made an assignment of funds for the work of the Committee, March 4th, 1914, and a contribution toward the expenses of the tests was received later by the American Railway Engineering Association from the United States Steel Corporation. Funds having been provided sufficient to make a beginning, the initial meeting of the Committee was held in Chicago, March 18th, 1914, at which there was a general discussion of the field of work of the Committee and of the way in which the work should be undertaken. A Sub-Committee was appointed to report on the methods of organization and scope of work of the Committee. At a meeting of the Committee held at Baltimore, June 3d, 1914, the report of this Sub-Committee was presented, discussed and adopted. Three Sub-Committees were appointed, (a) Executive, (b) Tests, and (c) Ways and Means. The programme of testing was referred to the Sub-Committee on Tests and it was decided that the first work should include the development of instruments and methods for tests on track, and that the preliminary tests on track should include observations of the equilibrium depression curve of the track under static load (the measurements to include rail, ties, ballast, and roadbed); and also measurements of the distri-

Report of Committee on Stresses in Railroad Track (continued).

bution of strains in the rail. The general plan of carrying on the work was referred to the Executive Committee. Mr. E. H. Fritch was elected Secretary of the Committee.

"During the summer and fall, plans have been developed and a beginning made. The work has included: (a) a study of the problem, (b) development of instruments and methods of carrying on the tests and trying these out in field and laboratory, and (c) static tests on railroad track with heavy locomotives and with single concentrated load. These tests were made on the track of the Illinois Central Railroad near Champaign, Ill. The Sub-Committee on Tests considered the results of the first of these field tests and adopted a general plan of procedure for the static tests. A test party was organized and a start was made on the field tests, and these tests were carried on until unfavorable weather interfered with the work. The work of computation, making records, and developing apparatus has since been continued. Enough experience has been gained to show that the methods used are practicable. During the winter it is expected that some laboratory tests will be made on ballast and on rail joints. Preliminary arrangements have been made for tests on the Baltimore and Ohio Railroad and on the Delaware, Lackawanna and Western Railroad. It is planned to carry on the test work during the coming season.

"The Engineering Experiments Station of the University of Illinois is co-operating with the Committee on this work, through the use of its laboratories and its staff, especially through the assistance of Professor H. F. Moore, in the development of apparatus and methods of tests. It is expected that this co-operative work will be of great assistance to the Committee.

"The Committee has found, as it expected, that the problem assigned to it is a very complicated one, indefinite in many ways, and containing many variables which must be eliminated or their influence determined. The experimental work will involve much tedious and painstaking work, and it is felt that results of value may be obtained only after prolonged work on the problem. The Committee feels that encouraging progress has been made, and enough has been done to warrant the expectation that on some parts of the problem, at least, information of value will be obtained.

"Respectfully submitted,
"Special Committee to Report on Stresses
in Railroad Track.
"A. N. Talbot,
"Chairman."

The President.—Is there any discussion on the report just read by Chairman Talbot? If not, and there is no objection, it will be accepted as a Progress Report.

We have now reached the period at which we will hear announcements from the Secretary, if he has any.

Rules for Government of Special Committees.

The Secretary.—Of interest to the membership, Mr. President, I think I should read the rules which have been adopted by the Board

of Direction for the appointment, government and procedure of Special Committees.*

Engineering Foundation

Mr. President, an invitation has been forwarded to all members of the Society, reading as follows:

"The Trustees of
The United Engineering Society
representing
The American Institute of Mining Engineers
The American Society of Mechanical Engineers
The American Institute of Electrical Engineers
and with the co-operation of
The American Society of Civil Engineers
request the honor of your presence
at the Inauguration of
The Engineering Foundation
on the evening of Wednesday the twenty-seventh of January
One thousand nine hundred and fifteen
at eight thirty o'clock
in the Auditorium of the Engineering Societies Building
Twenty-nine West Thirty-ninth Street
New York"

This was sent out before it was possible to make an explanation in type of the nature of this Trust and of the programme of the Inauguration Meeting.

The Engineering Foundation is the name given to a Fund to be "devoted to the advancement of the Engineering Arts and Sciences in all their branches, to the greatest good of the Engineering Profession and to the benefit of Mankind."

The initial gift for the Engineering Foundation is from a noted Engineer whose name will be announced at the Inauguration ceremonies.†

The administration of this Fund will be entrusted to a Board of eleven members elected by the Trustees of the United Engineering Society, of which two members will be elected from each of the four National Societies. The President of the United Engineering Society will, *ex officio,* be a member of the Board, and there will be two members chosen at large.

It is hoped that there will be a large attendance of the Profession at the meeting, at which the speakers will be Mr. Gano Dunn, President of the United Engineering Society and Past-President of the American Institute of Electrical Engineers; Dr. Henry Pritchett, President of the Foundation for the Advancement of Teaching; Dr.

* Printed in *Proceedings* for January, 1915, p. 7.

† Since this announcement was made, the Inaugural Meeting has been held and the donor stated to be Mr. Ambrose Swasey, of Cleveland, Ohio.

Engineering Foundation (continued).

Robert W. Hunt, Past-President of the American Institute of Mining Engineers; and Dr. Alexander C. Humphreys, Past-President of the American Society of Mechanical Engineers.

Ladies will be cordially welcomed at the meeting.

Annual Convention.

Mr. President, the Annual Convention of the Society will be held in San Francisco, Cal., September 16th to 18th, 1915. This covers the Thursday, Friday, and Saturday, immediately preceding the opening of the International Engineering Congress on September 20th, 1915. The date was chosen in order to enable members of the Society to attend both the Annual Convention and the Congress, and also to visit the Panama-Pacific Exposition, with the least possible expenditure of time. Thus it will be possible to make the trip from the East, attend all the meetings, excursions, etc., connected with both these meetings, and return within a period of three weeks.

The St. Francis Hotel in San Francisco has been selected as the headquarters' hotel, and a number of rooms have been reserved, both there and in the Palace Hotel. There are also a number of other hotels in which accommodations will be available at somewhat less cost.

Each of the three other National Societies has determined to hold a meeting at or about the same date, and it is proposed to make arrangements for a special train leaving New York about September 9th, and scheduled to arrive in San Francisco on the 15th. It is intended that the members of all these Societies, as well as such European Engineers as may attend the Congress, shall have the opportunity of going together on this train.

It is expected within a short time to issue to the 27 000 members of these four Societies a circular giving preliminary information as to the proposed special train, hotel rates, etc., which will enable those who desire to make their hotel reservations in ample time.

The President.—We have now reached the heading of New Business.

Resolution Relating to Opening Ceremonies of the Panama Canal.

Mr. Ockerson.—I have a resolution that I would like to offer:

"*Whereas:* Many distinguished members of the American Society of Civil Engineers, including six Past-Presidents, have been prominently identified officially with the several Canal Commissions and are largely responsible for the location, development of plans, and inauguration of construction work, all of which have resulted in a successful Isthmian Canal, and

"*Whereas:* The Chief Engineer of the Canal and Governor of the Canal Zone and his chief associates, whose energy and ability have carried to successful completion the greatest of all engineering works, are also members of this Society, therefore be it

"*Resolved:* That this Society should be formally represented at the opening ceremonies of the Panama Canal by a suitable delegation of its members, and that official sanction and recognition for such participation be requested from the proper authorities in charge thereof; and be it further

"*Resolved:* That the Board of Direction be and is hereby requested to take such steps as may be found necessary to carry these resolutions into effect."

Resolution seconded.

THE PRESIDENT.—Gentlemen, you have heard the resolution. The chair will be glad to hear any discussion on it. From the manner in which it was received, I assume that the vote will be unanimous. All in favor of the resolution will say "aye"; all opposed, "no".

It is carried.

I would ask that Professor Marx address the meeting on the subject of the International Engineering Congress, to be held in San Francisco.

International Engineering Congress.

CHARLES D. MARX, PRESIDENT-ELECT, AM. SOC. C. E.—Mr. President, and Members of the Society: Members who are organizing the International Engineering Congress have asked me to present in as few words as possible the claims of the International Engineering Congress for your support. Let me state briefly that when it was known that an International Fair was to be held in San Francisco in commemoration of this piece of engineering to which Mr. Ockerson has just referred, the engineers on the Pacific Coast felt that some special recognition of the work of the engineers should be shown.

The Commission in charge of the Fair was approached, and the members were asked whether it was likely that any support could be given to that undertaking. This support was refused. Thereupon, the engineers on the Coast made up their minds that, independent of those in charge of the Fair, they would arrange for an Engineering Congress, international in scope, to celebrate suitably the completion of this engineering work; and, as a beginning, the members of the Profession subscribed a fund of $10 000 for starting this work. We felt, of course, that this amount was far too small to carry out the object, and we then turned to our brothers in the East, to our fellow-members in the Eastern Associations, and through the hearty cooperation of the Secretary of this Society, and those of the other Engineering Societies, a guarantee fund was established which enabled us to go ahead with the work.

A local committee was organized, and I know that all of you have received the circulars which outline the scope of the work. A number of volumes, which we believe will be of high professional interest, will be published.

The expenses have been large, of course. We had counted on a large participation on the part of the engineers from foreign countries, and for reasons well known to all of you, we will probably be disappointed. That means that the Congress which it was intended should be very largely international, will be so to a somewhat less extent; but at the same time we have assurances of papers from those

International Engineering Congress (continued).

of our colleagues who are not otherwise engaged. The duty of supplementing this lacuna will fall, therefore, in a larger measure upon you, the members of our American Associations.

A special appeal has been made by Mr. A. M. Hunt, member of this Society and Chairman of the Publication Committee, to a number of the engineers in this country to furnish additional papers. We feel sure that such contributions will be equal in value, to all of you as members of the Profession, to those which might have been furnished by our foreign members.

I have in my pocket a statement of the number of members who, up to the present time, have joined the Congress.

Mr. Cattell, also a valued member of this Society, has sent you circulars, and has called attention to the fact that the membership fee in the Congress, which carries with it the receipt of one of the volumes of the *Transactions,* is only $5.

The first volume, which deals with the work carried out on the Panama Canal, will undoubtedly be one which each of you ought to feel proud to have in your engineering library. I think it is only necessary for me to read the list of subscriptions received to convince some of you that, unfortunately, in the rush of business, because of excessive employment, you have overlooked your duty of joining this Congress.

Let me state that the number of engineering and technical societies which have been corresponded with for participation in the Congress, is 325. Out of this total, 230 have responded. These societies have appointed delegates: in the United States, three; from Great Britain there is to be one; from Hungary, one. The total number of subscriptions to date is 2 361. I wish I might be able to add a cipher beyond that 1. 1 598 from the United States and possessions and 663 from foreign countries. 649 members of the American Society of Civil Engineers are enrolled in this Congress, scarcely 10% of our membership. 48 foreign countries are represented in the membership, and the following have the largest number of members: Canada, 86; Great Britain, 79; Holland, 26; France, 56; Germany, 59; Brazil, 30; Australia, 34; Japan, 34; Russia, 22; the Argentine Republic, 20; and India, 20.

The enrollment in the United States from States credited with more than 20 members is as follows: California, 352; New York, 326; Pennsylvania, 135; Illinois, 83; Massachusetts, 68; Ohio, 48; New Jersey, 41; Washington on the Coast, 35; Washington, D. C., 34; Oregon, 31; Missouri, 30; Colorado, 27; Michigan, 25; and Virginia, 20.

Approximately 320 papers were expected before the declaration of war; but at the present date only 225 may be counted on, although there is still the hope of securing a few from certain countries abroad.

The participation of foreign Societies from the warring zone is

almost an impossibility, although a request has been received from at least one Society in England, asking information as to the steps to be taken to appoint delegates to the Congress.

Contributions from abroad have already been received. I will not bother you with the list of those, but will state that they are coming in. Enrollment in the Congress is proceeding very actively, considering the present condition; and if active work is kept up, we hope to have an enrollment of 4 000.

I make a special plea on behalf of the International Engineering Congress, that you support it to the best of your ability. I assure you that if you come out there, and I hope many of you will, you will be met with the most hearty welcome. I thank you.

THE PRESIDENT.—The time has now arrived for your President to surrender his gavel to his successor. I will appoint Past-Presidents C. C. Schneider and Charles Macdonald, a Committee to conduct our new President to the Chair.

Gentlemen, I have the honor of introducing our new President, Mr. Charles D. Marx. President Marx takes the Chair.

PRESIDENT MARX.—Members of the Society: With a full appreciation of the high honor conferred, I thank you. Counting upon the hearty co-operation of the members of the Board of Direction, and relying upon their wisdom and experience, I propose to further the interests of this Society to the best of my ability.

The Secretary has an announcement to make.

THE SECRETARY.—The President wishes me to announce that a meeting of the Board of Direction will be held at 2 o'clock in the Board Room, and asks that all members of the Board be there as promptly as possible at that hour. Directly after adjournment, luncheon will be served.

THE PRESIDENT.—Is there any more business to come before us at the present time?

THE SECRETARY.—Not that I have, sir.

THE PRESIDENT.—If not, a motion to adjourn is in order. We stand adjourned. Adjourned.

EXCURSIONS AND ENTERTAINMENTS AT THE SIXTY-SECOND ANNUAL MEETING

Wednesday, January 20th, 1915.—After the Business Meeting, lunch for about 800 members was served at 1 P. M. at the Society House.

At 3 P. M., through the courtesy of the Public Service Commission for the First District, Alfred Craven, M. Am. Soc. C. E., Chief Engineer, and the contractors for Sections 11, 12, and 13 of the new subway on Lexington Avenue, Manhattan, about 400 members made an inspection trip from 93d to 129th Streets.

At 9 P. M., there was a Reception, with dancing, at the Hotel Astor, at which the attendance was 717.

Thursday, January 21st, 1915.—The day was devoted to an excursion over a portion of the Long Island Railroad. A party of about 570 left the Pennsylvania Station at 9.45 A. M. on a special train which passed through the East River Tunnel to the Sunnyside Yard, and thence to Winfield Junction (where an extensive cut-off is being constructed), and then to Flushing and to Great Neck, the elimination of a number of grade crossings being viewed *en route*. The train then returned to Winfield Junction and switched south to Jamaica where the extensive improvements in connection with the new Jamaica Terminal were inspected.

From Jamaica the train returned to Long Island City over the Montauk Division to Glendale Junction, and thence over the Glendale Cut-off to the Main Line and west to the Montauk Cut-off, crossing above the portals of the East River Tunnels, from which point an interesting view of Sunnyside Yard was obtained. The train then proceeded *via* the Montauk Division to Bushwick Junction, where a four-track depression of the railroad is under way to eliminate grade crossings.

At Bushwick Junction, by invitation of W. H. Gahagan, M. Am. Soc. C. E., the party had lunch at "Trommers", and subsequently inspected the work being done in that locality.

The party left East New York at 2.30 P. M., and *en route* to Bay Ridge, a stop was made to inspect the reconstruction of the Sea Beach Railway for the New York Municipal Railway Company as a four-track depressed line from its connection with the Rapid Transit Subway at 65th Street and Fourth Avenue, Brooklyn, to a point south of 86th Street, a distance of about 4¼ miles.

Proceeding to Bay Ridge, the party was conveyed by ferry to the Long Island Railroad Dock at East 34th Street, Manhattan, arriving at about 5 P. M.

In the evening, at the Society House, at 8 P. M., there was a Lecture by Professor Lawrence D. Kitchell, on the Glacier National Park,

Montana, illustrated with lantern slides and motion pictures. This was followed by a social and informal Smoker, at which the attendance was about 860.

The following list contains the names of 927 members of various grades who registered as being in attendance at the Annual Meeting. The list is incomplete, as some members failed to register, and it does not contain the names of any of the guests of the Society or of individual members. The number of ladies and other guests is estimated at about 700.

Abbott, C. P..White Plains, N. Y.
Abbott, Hunley...New York City
Ackerman, J. W...Auburn, N. Y.
Addis, W. G.....Winthrop, Mass.
Aiken, W. A.....New York City
Aims, W. I.....New York City
Alderson, A. B...Hartford, Conn.
Alexander, H. J..New York City
Allen, E. Y.....Orange, N. J.
Allen, F. W..White Plains, N. Y.
Allen, H. D.....Newark, N. J.
Allen, Kenneth...New York City
Ammann, O. H.,
New Brighton, N. Y.
Andrew, C. R.,
New Matamoras, Ohio
Appel, H. A.....New York City
Armstrong, R. S..Brooklyn, N. Y.
Arnold, W. H....New York City
Ashley, Carl.....New York City
Ashmead, P. H...New York City
Atkinson, Asher,
New Brunswick, N. J.
Atwood, T. C..New Haven, Conn.
Auryansen, Frederick,
Jamaica, N. Y.
Ayres, Q. C.....Columbus, Miss.

Babcock, W. S....New York City
Baird, H. C.....New York City
Baldwin, W. J...Brooklyn, N. Y.
Ball, L. A.....New York City
Ballinger, W. F..Philadelphia, Pa.
Bamford, W. B....Belmar, N. J.
Barnes, M. G.....Albany, N. Y.
Barnes, O. F...Jersey City, N. J.
Barnes, W. E.....Boston, Mass.
Barnett, R. P.....New York City
Barney, P. C.....Brooklyn, N. Y.
Barney, W. J.....New York City
Bascome, W. R....New York City
Basinger, J. G....New York City
Baum, George.....Yonkers, N. Y.
Bean, G. L.....Philadelphia, Pa.
Beaty, R. E.....New York City
Becker, E. J...Schenectady, N. Y.
Becker, R. C.....New York City
Beebe, H. R.....Utica, N. Y.
Beekman, J. V., Jr..Boston, Mass.
Beggs, G. E.....Princeton, N. J.
Belden, E. T.....Pittsfield, Mass.
Belknap, F. W.....New York City
Belknap, J. M...Manhasset, N. Y.
Bendel, Jacob.....New York City
Benedict, F. N.,
Mount Vernon, N. Y.
Bensel, J. A...Bernardsville, N. J.
Bentley, J. C.....Cambridge, Md.
Berger, Bernt.....New York City
Berger, John.....New York City
Bettes, C. R..Far Rockaway, N. Y.
Betts, R. T.....New York City
Bevan, L. J.....New York City
Bigelow, W. W..Springfield, Mass.
Bilyeu, C. S.....New York City
Blackmore, G. G..New York City
Blair, Alexander...Summit, N. J.
Blair, C. M....New Haven, Conn.
Blakeley, G. H.,
South Bethlehem, Pa.

Blakeslee, Clarence,
New Haven, Conn.
Blakeslee, H. L.,
New Haven, Conn.
Blanchard, A. H..New York City
Blatch, N. S.....New York City
Blatt, Max.....New York City
Boardman, C. S....Buffalo, N. Y.
Boardman, W. H...Newark, N. J.
Boes, F. C.....New York City
Bogart, John.....New York City
Bogert, C. L.....New York City
Bogue, V. G.....New York City
Bolger, E. G.....Altoona, Pa.
Boller, A. P., Jr.,
East Orange, N. J.
Bond, G. M.....Hartford, Conn.
Boniface, Arthur..Scarsdale, N. Y.
Bontecou, Daniel,
Kansas City, Mo.
Booth, G. W.....New York City
Booz, H. C.....Philadelphia, Pa.
Bortin, Harry....Brooklyn, N. Y.
Boucher, W. J.....New York City
Bouton, H. R.....New York City
Bowditch, J. H.,
New Brighton, N. Y.
Bowman, A. L....New York City
Boyd, R. W.....New York City
Brace, J. H..Cedars, Que., Canada
Brackett, Dexter....Boston, Mass.
Bradbury, R. R.,
East Pleasantville, N. Y.
Bradley, F. E.....New York City
Brainerd, H. A....Westfield, N. J.
Bramwell, G. W..New York City
Braslow, Barnett..New York City
Brendlinger, P. F.,
Philadelphia, Pa.
Brennan, J. G.....Albany, N. Y.
Brennan, J. L.....New York City
Breuchaud, Jules..New York City
Briggs, J. A.....New York City
Briggs, W. C.....Brooklyn, N. Y.
Britton, G. C..Brockwayville, Pa.
Brooks, J. P.....Potsdam, N. Y.
Brown, R. H.....New York City
Brown, S. P.,
Montreal, Que., Canada
Brush, W. W.....Brooklyn, N. Y.
Bryan, C. W.....New York City
Buck, R. S.....New York City
Buettner, O. G. H..New York City
Burdett, F. A.....New York City
Burgess, G. H.....Albany, N. Y.
Burnham, F. W....New York City
Burpee, G. W.....New York City
Burpee, Moses.....Houlton, Me.
Burr, W. H.....New York City
Burroughs, H. R..New York City
Bush, E. W.....Hartford, Conn.
Bush, Lincoln..East Orange, N. J.
Buss, A. S.....High Falls, N. Y.
Butler, M. J.,
Montreal, Que., Canada

Cadwallader, W. L.,
New York City
Caldwell, F. E.....Newton, N. J.
Cappelen, F. W.,
Minneapolis, Minn.
Carey, E. G...White Plains, N. Y.
Carmalt, L. J....Hartford, Conn.
Carpenter, A. W..Yonkers, N. Y.
Carpenter, C. E....Yonkers, N. Y.
Carr, Albert....East Orange, N. J.
Carstarphen, F. C..Trenton, N. J.
Casani, A. A.....New York City
Catlin, H. B.....New York City
Chambers, R. H..Flushing, N. Y.
Chappell, T. F....New York City
Chase, C. F...New Britain, Conn.
Christian, G. L....New York City
Church, E. C.....New York City
Church, F. B...Great Neck, N. Y.
Claflin, W. B.,
West Redding, Conn.
Clapp, O. F.....Providence, R. I.
Clapp, S. K.....Ashokan, N. Y.
Clark, A. E..White Plains, N. Y.

Clarke, E. W..Pleasantville, N. Y.
Clarke, G. C.,
Richmond Hill, N. Y.
Clement, S. B.,
North Bay, Ont., Canada
Closson, W. G....Brooklyn, N. Y.
Codwise, H. R....Brooklyn, N. Y.
Cohen, A. B.....Hoboken, N. J.
Cohen, F. W.,
Upper Montclair, N. J.
Cole, E. S.....New York City
Cole, G. N.....New York City
Coleman, J. F...New Orleans, La.
Collier, B. C.....New York City
Coltman, Robert, Jr.,
Elmhurst, N. Y.
Comber, S. X.....New York City
Condron, T. L.....Chicago, Ill.
Connell, H. L.....New York City
Connell, W. H...Philadelphia, Pa.
Connelly, J. A. A..New York City
Connor, E. H.Leavenworth, Kans.
Constant, F. H...Princeton, N. J.
Cook, F. S.....Yonkers, N. Y.
Cook, J. H.....Paterson, N. J.
Coombs, R. D.....New York City
Coombs, S. E.....Yonkers, N. Y.
Cooper, C. W.....New York City
Cooper, D. R.....Caldwell, Ohio
Corning, D. T.....Troy, N. Y.
Corthell, E. L.,
North Egremont, Mass.
Coyne, H. L....Woodhaven, N. Y.
Crane, A. S.....New York City
Crane, C. A.....New York City
Crane, F. E....Amsterdam, N. Y.
Crehore, W. W....New York City
Cresson, B. F., Jr.,
Jersey City, N. J.
Creuzbaur, R. W..Brooklyn, N. Y.
Crocker, H. S.....Denver, Colo.
Crocker, J. R.....New York City
Crooks, C. H.....New York City
Crosby, W. W.....Baltimore, Md.
Crowell, F. S.....Flushing, N. Y.
Crowell, Foster....New York City
Cuddeback, A. W..Paterson, N. J.
Cullen, J. F.....Philadelphia, Pa.
Culver, T. C..Purdy Station, N. Y.
Cummings, Noah..New York City
Cummings, R. A..Pittsburgh, Pa.
Curtis, C. E.....Cleveland, Ohio
Curtis, F. S.....Boston, Mass.
Cushing, W. C....Pittsburgh, Pa.
Cutler, L. G...East Orange, N. J.

Dailey, J. A...East Orange, N. J.
Dakin, A. H., Jr..New York City
Dallis, P. A.....Atlanta, Ga.
Dana, R. T.....New York City
Davies, J. P.....New York City
Davies, J. V.....Flushing, N. Y.
Davis, B. H.....New York City
Davis, J. L.Manchester Center, Vt.
Dean, A. W.....Boston, Mass.
de Mey, E. J. B..Cincinnati, Ohio
Develin, R. G....Philadelphia, Pa.
Devlin, H. S.....Newark, N. J.
de Wyrall, Cyril,
Ridgefield Park, N. J.
Deyo, S. L. F.....New York City
Diamant, A. H....New York City
Dibert, H. McM.....Troy, N. Y.
Diebitsch, Emil.....Nutley, N. J.
Dimon, D. Y.....New York City
Dixon, G. G.....Kent, Ohio
Dodge, S. D.....Brooklyn, N. Y.
Donham, B. C..Glen Ridge, N. J.
Donle, E. R.....Belmont, Mass.
Doolittle, F. W....New York City
Doron, C. S.....Brooklyn, N. Y.
Dougherty, R. E.,
White Plains, N. Y.
Douglass, A. C.,
Niagara Falls, N. Y.
Doyen, G. E.....New York City
Drayton, Newbold,
Philadelphia, Pa.
Drew, C. D.....Flushing, N. Y.
Dunn, H. L...New London, Conn.

Durant, Aldrich...New York City
Durfee, J. J.....Brooklyn, N. Y.
Durham, H. W....New York City
Durham, Leicester.Scarsdale, N. Y.
Dykeman, C. F...Brooklyn, N. Y.

Earle, Thomas.....Steelton, Pa.
Easterbrook, F. J.,
New Haven, Conn.
Eckersley, J. O....New York City
Eddy, H. P.....Boston, Mass.
Edmondson, R. S..New York City
Edwards, D. G....Brooklyn, N. Y.
Edwards, J. H.....Passaic, N. J.
Ehrbar, L. H.....Newark, N. J.
Eide, Torris.....New York City
Ellis, H. C....White Plains, N. Y.
Ellis, J. W.....Woonsocket, R. I.
Elmer, H. N.....Chicago, Ill.
Ely, G. W., Jr.....New York City
Emerson, K. B....New York City
Endicott, M. T.Washington, D. C.
Esselstyn, H. H... Detroit, Mich.
Ewing, W. W.....Westfield, N. J.

Farley, G. P...Garden City, N. Y.
Farley, M. M.....New York City
Farnham, A. B....Pittsfield, Mass.
Farrington, H. P..New York City
Fay, F. H.....Boston, Mass.
Feigel, J. H.....Buffalo, N. Y.
Feiner, M. A.....New York City
Ferguson, L. R..Philadelphia, Pa.
Ferry, C. A....New Haven, Conn.
Fetherston, J. T...New York City
Files, T. H.....New York City
Finch, J. K.....New York City
Firth, E. W.....New York City
Fischer, Charles, Jr.New York City
Fisher, E. A.....Rochester, N. Y.
Fisher, H. T.,
Montreal, Que., Canada
Fitch, C. L.....Chicago, Ill.
Fletcher, Robert...Hanover, N. H.
Flinn, A. D.....Yonkers, N. Y.
Ford, H. C.....New York City
Ford, W. H.....Philadelphia, Pa.
Fort, E. J.....Brooklyn, N. Y.
Foss, F. E.....New York City
Foster, E. H.....New York City
Fox, W. F.....New York City
Franklin, C. M....New York City
Freeman, M. H...Brooklyn, N. Y.
French, A. W....Worcester, N. Y.
French, Halsey..Brooklyn, N. Y.
French, J. B.....New York City
Frost, G. H.....Plainfield, N. J.
Frost, G. S.....Brooklyn, N. Y.
Fuller, G. W.....New York City
Fuller, W. B.....Paterson, N. J.
Fuller, W. E.....New York City

Gagel, Edward.West Haven, Conn.
Gahagan, W. H...Brooklyn, N. Y.
Gardiner, F. W....Yonkers, N. Y.
Gardner, Warren..New York City
Garfield, C. A.White Plains, N. Y.
Gartensteig, Charles,
New York City
Gaston, L. P.....New York City
Gay, H. S.....Bayonne, N. J.
Gerhard, N. P....Scarsdale, N. Y.
Gifford, G. E.....New York City
Gildersleeve, A. C.New York City
Giles, J. A....Binghamton, N. Y.
Giles, Robert.....New York City
Gillen, W. J.....New York City
Gillespie, R. H....New York City
Gilman, Charles...New York City
Glover, P. H.....Harrington, Me.
Godfrey, Hollis..Philadelphia, Pa.
Goldmark, Henry..New York City
Goldsborough, J. B.New York City
Goldsmith, William.New York City
Gons, L. R.....Wilmington, Del.
Goodell, J. M....Worcester, Mass.
Goodman, Joseph..New York City
Goodman, Louis...New York City
Goodsell, D. B....Brooklyn, N. Y.
Goodwin, R. E.....New York City

Govern, E. J....Rochester, N. Y.
Gowen, J. F.....Ossining, N. Y.
Graham, G. A.....New York City
Granbery, J. H....New York City
Gray, Edward.....Richmond, Va.
Gray, William.....New York City
Greathead, J. F.,
White Plains, N. Y.
Green, C. N.....New York City
Greene, Carleton..New York City
Greenlaw, R. W...New York City
Gregg, J. H. C....Brooklyn, N. Y.
Gregory, J. H.....New York City
Gregory, W. B..New Orleans, La.
Greiner, J. E.....Baltimore, Md.
Guttridge, J. A....New York City

Haas, P. L.....New York City
Hale, H. M.....Brooklyn, N. Y.
Hall, J. L.....Seattle, Wash.
Hall, M. W.....New York City
Hall, W. H....New Britain, Conn.
Hallock, J. C.....Newark, N. J.
Halsey, W. H.,
Bridgehampton, N. Y.
Hamilton, J. W....New York City
Hammel, E. F.....New York City
Hammond, C. A.,
Mount Vernon, N. Y.
Hammond, G. T..Brooklyn, N. Y.
Hammond, J. F.,
Richmond Hill, N. Y.
Hanavan, W. L.,
Long Island City, N. Y.
Hanmer, H. J..Gloversville, N. Y.
Hansel, Charles...Cranford, N. J.
Hansen, Paul.....Urbana, Ill.
Harby, Isaac.Richmond Hill, N. Y.
Harding, H. S.....New York City
Haring, Alexander.New York City
Harps, H. M..Perth Amboy, N. J.
Harrington, J. L.Kansas City, Mo.
Harris, B. B.....New York City
Harris, F. R.....Brooklyn, N. Y.
Harris, G. H.....Detroit, Mich.
Harris, H. F.....Trenton, N. J.
Harte, C. R....New Haven, Conn.
Hartman, A. H....Baltimore, Md.
Hartmann, E. F...New York City
Harwi, S. J.....Bayonne, N. J.
Harwood, G. A....New York City
Hasbrouck, Oscar..Hudson, N. Y.
Haskell, E. E.....Ithaca, N. Y.
Haskins, W. J....New York City
Hauck, William...New York City
Hawley, J. B....Fort Worth, Tex.
Hazelton, C. W.,
Montague City, Mass.
Hazen, Allen.....New York City
Hazen, R. W....Whitefield, N. H.
Hazen, W. N.....Orange, N. J.
Healy, J. P....Washington, D. C.
Healy, J. R.....New York City
Heilbronner, L. C....Utica, N. Y.
Heiser, A. B.....New York City
Heiser, W. J.....New York City
Heller, J. W.....Newark, N. J.
Henry, P. W.....New York City
Hermanns, F. E...New York City
Herschel, Clemens.New York City
Hewes, V. H.....New York City
Hewett, B. H. M...New York City
Hewitt, George....New York City
Higgins, C. H.....New York City
Higgins, J. W..Roselle Park, N. J.
Hildreth, J. L., Jr.,
Jersey City, N. J.
Hilton, J. C.....New York City
Hinman, L. R.....Denver, Colo.
Hitchcock, F. C....New York City
Hoag, S. W., Jr....New York City
Hodgdon, F. W.....Boston, Mass.
Hodge, H. W.....New York City
Hogan, J. P..Tompkinsville, N. Y.
Holbrook, A. R....Brooklyn, N. Y.
Holbrook, Percy...New York City
Holden, C. A.....Hanover, N. H.
Holden, C. A.Mount Vernon, N. Y.
Holdredge, N. C...New York City
Holland, C. M....Brooklyn, N. Y.

Holtzman, S. F.,
Hastings-on-Hudson, N. Y.
Hone, A. C.....New York City
Honness, G. G....Ashokan, N. Y.
Hoover, A. P.....Paterson, N. J.
Horton, R. E.....Albany, N. Y.
Hough, D. L....Petrograd, Russia
Hough, U. B.....Spokane, Wash.
Houston, J. J. L..Jamaica, N. Y.
Houston, W. O.....Jackson, Mich.
Hovey, O. E.....New York City
Howard, L. T.....New York City
Howard, O. Z.....New York City
Howell, W. A.....Newark, N. J.
Howes, C. P.,
South Yarmouth, Mass.
Howes, D. W.....Brooklyn, N. Y.
Hubbard, Daniel..Baltimore, Md.
Hubbard, W. D.,
Brown Station, N. Y.
Hubbell, G. S....New York City
Hudson, C. W....Montclair, N. J.
Hudson, H. W.....New York City
Hulsart, C. R.....New York City
Humphrey, R. L.Philadelphia, Pa.
Hunt, A. M..San Francisco, Cal.
Hunt, C. E.....New York City
Hunt, Charles Warren,
New York City
Hurd, H. L...White Plains, N. Y.
Hurlbut, C. C....Montclair, N. J.
Hurley, J. P.....Brooklyn, N. Y.
Hyde, H. E.....Ithaca, N. Y.
Hyde, J. L.....Westfield, Mass.

Ilsley, A. B....Washington, D. C.
Isaacs, J. D.....New York City

Jackson, J. F..New Haven, Conn.
Jacobs, R. H.....New York City
Jarrett, E. S.....New York City
Jenkins, J. E.....New York City
Joachimson, Martin,
New York City
Johannesson, Sigvald,
Upper Montclair, N. J.
Jonah, F. G.....St. Louis, Mo.
Jones, R. S.....New York City
Jones, S. R.....New York City
Jonson, E. F.....New York City
Jordan, L. C.....Brooklyn, N. Y.
Just, G. A.Long Island City, N. Y.

Karner, W. J.....New York City
Kaufman, Gustave.New York City
Keefe, D. A.....Athens, Pa.
Keefer, C. H.Ottawa, Ont., Canada
Keith, H. C.....New York City
Kelley, W. D.....Yonkers, N. Y.
Kellogg, R. C.....Brooklyn, N. Y.
Kelly, C. W....New Haven, Conn.
Kelly, H. A.....Jersey City, N. J.
Khuen, Richard...Pittsburgh, Pa.
Kienle, J. A.....New York City
Kimball, F. C.....Summit, N. J.
Kimball, G. H.....Detroit, Mich.
King, E. T.....Arrochar, N. Y.
Kinney, W. M.....Chicago, Ill.
Kinsey, W. A.....Newark, N. J.
Kirchgraber, E. H.New York City
Kirchner, P. A....New York City
Kirkwood, H. C....New York City
Kittredge, G. W...Yonkers, N. Y.
Kittredge, H. C..Rochester, N. Y.
Knighton, J. A....New York City
Knowles, Morris...Pittsburgh, Pa.
Knox, S. B.....New York City
Kolb, H. J.....Brooklyn, N. Y.
Kornfeld, A. E....New York City
Krellwitz, D. W....New York City
Kuehnle, W. L....New York City

La Chicotte, H. A.Brooklyn, N. Y.
Lackey, O. F.....Baltimore, Md.
Lake, E. N.....Chicago, Ill.
Lamson, W. M....Brooklyn, N. Y.
Landreth, O. H.Schenectady, N. Y.
Lang, F. A.....New York City
Lange, T. F.....New York City

Langthorn, J. S....New York City
Lannan, L. E.Mount Vernon, N. Y.
Latimer, C. A.....Ambridge, Pa.
Lavis, Fred.....New York City
Lawrence, E. V....New York City
Lee, E. M.....New York City
Lee, W. B.....New York City
Leeds, C. T.....Los Angeles, Cal.
Leighton, George...Glenburn, Pa.
Leiser, Ferdinand, Jr.,
Brooklyn, N. Y.
Leonard, H. R...Philadelphia, Pa.
Leser, Henry.....New York City
Lesley, R. W.....Philadelphia, Pa.
Letson, T. H.....New York City
Lewis, E. W....New Haven, Conn.
Lewis, N. P.....New York City
Lex, W. I.....Philadelphia, Pa.
Lichtenstein, Harry,
New York City
Lindau, A. E.....Winnetka, Ill.
Lindenthal, Gustav..New York City
Lindholm, C. B...Pittsfield, Mass.
Linton, W. P.....New York City
Llewellyn, F. T....New York City
Lobo, Carlos.....Brooklyn, N. Y.
Loewenstein, Jacob.New York City
Logan, W. S.....Arlington, N. J.
Longley, F. F....New York City
Look, M. J...Brown Station, N. Y.
Loomis, Horace....New York City
Loomis, L. B.....New York City
Lorah, W. L.....Brooklyn, N. Y.
Loughran, J. F...Kingston, N. Y.
Lovell, E. B.....Montclair, N. J.
Loweth, C. F.....Chicago, Ill.
Lowinson, Oscar...New York City
Lucas, G. L.....New York City
Lundgren, Leonard.Portland, Ore.
Lundie, John.....New York City
Lynde, Clifford...Brooklyn, N. Y.
Lyons, H. C.....Flushing, N. Y.

MacCornack, C. W.,
Phœnixville, Pa.
Macdonald, Charles,
Brooklyn, N. Y.
MacFeeters, J. O.Glen Ridge, N. J.
MacGregor, R. A..New York City
Machen, H. B....New York City
Mack, P. W.....New York City
Macnab, G. T.....New York City
McComb, D. E..Washington, D. C.
McCormack, E. W.,
Washington, D. C.
McDonald, Hunter,
Nashville, Tenn.
McDowell, F. F....Yonkers, N. Y.
McEntire, Lloyd.Frenchtown, N. J.
McKenzie, T. H.,
Southington, Conn.
McLoughlin, F. O. X.,
New York City
McLure, N. R....Phœnixville, Pa.
McMinn, T. J.....New York City
McMullen, R. W.,
New Canaan, Conn.
McNab, William,
Montreal, Que., Canada
McNaugher, D. W.Pittsburgh, Pa.
McPherson, R. H..New York City
Madden, J. H.....New York City
Malmquist, E. A...New York City
Malukoff, A. J.....New York City
Manahan, E. G....New York City
Manley, L. B.....Boston, Mass.
Mann, J. L.....Rosebank, N. Y.
Mansfield, W. H.....Troy, N. Y.
Marsh, F. B.....Crestwood, N. Y.
Martin, B. C.Poughkeepsie, N. Y.
Martin, E. S.....New York City
Marx, C. D.,
Stanford University, Cal.
Mason, Francis....New York City
Matheson, J. D....New York City
Matlaw, I. S.....New York City
Mayper, Victor....New York City
Meadowcroft, William,
New York City
Meeker, R. A.....Plainfield, N. J.

Meggy, R. L. G....Fanwood, N. J.
Mehren, E. J.....New York City
Meigs, John.....Philadelphia, Pa.
Melius, L. L.....New York City
Menke, William...New York City
Merriman, Mansfield,
New York City
Merriman, R. M.,
Central Valley, N. Y.
Merriman, Thaddeus,
New York City
Metcalf, Leonard....Boston, Mass.
Michael, H. L.....Buffalo, N. Y.
Middlebrook, C. T...Albany, N. Y.
Miller, Frank.....Passaic, N. J.
Miller, Hugh.....Potsdam, N. Y.
Miller, H. A.....Boston, Mass.
Miller, M. M.....New York City
Miller, R. P.....New York City
Mitchell, S. P....Philadelphia, Pa.
Mogensen, O. E...Plainfield, N. J.
Molitor, F. A.....New York City
Monsarrat, C. N.,
Montreal, Que., Canada
Montfort, Richard..Louisville, Ky.
Moore, E. J.....Yonkers, N. Y.
Moore, F. F....Hawthorne, N. Y.
Moore, L. E.....Boston, Mass.
Moore, S. W..East Elmhurst, N. Y.
Moore, W. H...New Haven, Conn.
Morrill, G. P.,
Sagua la Grande, Cuba
Morrison, H. J..Amsterdam, N. Y.
Morrison, W. H., Jr.,
Brooklyn, N. Y.
Morse, C. F.....Watertown, N. Y.
Morse, C. M.....Buffalo, N. Y.
Morssen, C. M.,
Montreal, Que., Canada
Morton, W. S.....New York City
Moulton, O. M.....Gorham, Me.
Mozart, W. J.....Westboro, Mass.
Murphy, F. E.,
West New Brighton, N. Y.
Murphy, J. F..Tompkinsville, N. Y.
Murray, J. F....Philadelphia, Pa.
Myers, C. H.....New York City
Myers, J. H...White Plains, N. Y.

Neely, W. R.....Tarrytown, N. Y.
Nelson, A. H..Atlantic City, N. J.
Nelson, J. C.....New York City
Nelson, J. W.....New York City
Neuman, D. L.....New York City
Neumeyer, R. E....Bethlehem, Pa.
Newell, F. H..Washington, D. C.
Newhall, H. L.....New York City
Nial, W. A.....Troy, N. Y.
Nichols, C. H..New Haven, Conn.
Noble, F. C.....Brooklyn, N. Y.
Northrop, A. A....New York City

Oakley, G. I....Little Falls, N. Y.
Obreiter, J. W...Bloomfield, N. J.
O'Brien, J. H.,
Montreal, Que., Canada
Ockerson, J. A.....St. Louis, Mo.
Ockert, F. W.....New York City
O'Connell, G. P....Holyoke, Mass.
O'Connor, C. J....New York City
O'Connor, J. A.....Albany, N. Y.
Ogden, H. N.....Ithaca, N. Y.
Ogden, J. C.....New York City
Okeson, W. R.....Orange, N. J.
Olmsted, A. E.....New York City
Olmsted, H. W....New York City
Opdyke, S. B., Jr...New York City
O'Rourke, J. F....New York City
Orrok, G. A.....New York City
Ott, S. J.....Rutherford, N. J.
Overocker, D. W.....Troy, N. Y.
Owen, James.....Newark, N. J.

Paddock, H. C.....New York City
Palmer, S. B.....Norwich, Conn.
Parker, C. J.....Bronxville, N. Y.
Parker, J. L.....New York City
Parker, W. P.....New York City
Parlin, R. W.....Brooklyn, N. Y.

Parmley, W. C.,
Upper Montclair, N. J.
Parsons, H. de B. .New York City
Peabody, W. W.,
White Plains, N. Y.
Pegram, G. H. . . . .New York City
Pemoff, J. J. . . . . .New York City
Pendlebury, Edward,
Arlington, N. J.
Perrine, George. . . .New York City
Perrine, Harold. . . .New York City
Perry, F. W. . . . . .Brooklyn, N. Y.
Perry, J. P. H. . . .New York City
Pfau, J. W. . . . . .New York City
Philips, G. W. . . . .Flushing, N. Y.
Philips, J. H. . . .Glen Ridge, N. J.
Pierce, F. E. . . . . .New York City
Pirnie, H. M. . . .Springfield, Mass.
Pollock, C. D. . . . . .New York City
Pomeroy, L. R. . . . .New York City
Pommerer, R. W.,
Mount Vernon, N. Y.
Pond, F. H. . . . . .Brooklyn, N. Y.
Pond, H. O. . . . . .Tenafly, N. J.
Poole, C. A. . . . . .Rochester, N. Y.
Porter, H. G. . . . . .New York City
Porter, J. E. . . . . .Yonkers, N. Y.
Porter, J. M. . . . . .Easton, Pa.
Potter, Alexander. .New York City
Potts, Clyde. . . .Morristown, N. J.
Powelson, W. V. N. .New York City
Powers, C. V. V. . .New York City
Pratt, A. H. . .White Plains, N. Y.
Pratt, F. E. . . . . .Glen Ridge, N. J.
Preston, G. H. . . . . .New York City
Preston, H. W. . . . . .Elmira, N. Y.
Price, Frank. . . . . .Glen Cove, N. Y.
Price, F. O. . . . . .Brooklyn, N. Y.
Priest, B. B. . . .East Orange, N. J.
Proctor, R. F. . . . . .Baltimore, Md.
Purver, G. M. . . . . .Brooklyn, N. Y.

Quimby, H. H. . .Philadelphia, Pa.
Quimby, J. H. .Pearl River, N. Y.
Quincy, Edmund. .New York City
Quinn, M. F. . . . . .New York City

Rankin, E. S. . . . . .Newark, N. J.
Ray, G. J. . . . . .East Orange, N. J.
Reed, H. B. . . . . .New York City
Reeves, W. F. . . . . .New York City
Reid, H. A. . . . . .Jamaica, N. Y.
Rennell, H. H. . . . .New York City
Requardt, G. J. . . . . .Baltimore, Md.
Reynolds, A. M. .Glen Ridge, N. J.
Reynolds, J. O. . . .Brooklyn, N. Y.
Rhett, A. H. . . . . .Brooklyn, N. Y.
Rice, G. S. . . . . .New York City
Richardson, Clifford,
New York City
Richardson, J. D. . . .Corona, N. Y.
Richardson, J. H. . . .Boston, Mass.
Richardson, T. F.,
Rutherford, N. J.
Ridgway, Robert. . .New York City
Riedel, J. C. . . . . .Brooklyn, N. Y.
Rights, L. D. . . . . .New York City
Ripley, H. L. . . . . .Brockton, Mass.
Ripley, Joseph. . . . . .Albany, N. Y.
Ripley, T. M. . . .Watertown, N. Y.
Robbins, F. H. . . . . .New York City
Roberts, R. F. . . . . .Nutley, N. J.
Robinson, E. F. . . .New York City
Robinson, G. L. . . .New York City
Robinson, Harry,
West Haven, Conn.
Rockwood, N. C. . .Brooklyn, N. Y.
Rogers, A. W. . . . . .Rochester, N. Y.
Rogers, E. H. .West Newton, Mass.
Rogge, J. C. L. . . .Katonah, N. Y.
Rohrer, Grant. . . . . .New York City
Rosenberg, Friedrich,
Montclair, N. J.
Rothrock, W. P. . .New York City
Rugg, W. F. . . . . .Peekskill, N. Y.
Rust, H. P. . . . . .New York City
Ryan, M. H. . . . . .New York City

Sackett, A. J. . . . . .New York City
Sanborn, F. B. .Tufts College, Mass.

Sanborn, J. F....New York City
Sargent, E. H.....Albany, N. Y.
Saville, C. M....Hartford, Conn.
Schall, F. E..South Bethlehem, Pa.
Schermerhorn, H. O..Troy, N. Y.
Schmidt, H. H...Brooklyn, N. Y.
Schneeweiss, A. E..Newark, N. J.
Schneider, C. C..Philadelphia, Pa.
Schroeder, F. C..New York City
Schwarze, C. T.,
South Orange, N. J.
Scorgie, J. C....Cambridge, Mass.
Seabury, G. T.White Plains, N. Y.
Seage, C. E..New Brighton, N. Y.
Seaman, H. B....Brooklyn, N. Y.
Searle, C. D.....New York City
Seaver, Clifford...New York City
Selby, O. E.....Cincinnati, Ohio
Selmer, W. L....New York City
Serber, D. C.....New York City
Shailer, R. A....Brookline, Mass.
Shaw, G. H....Washington, D. C.
Sheridan, L. V....New York City
Sherman, C. W.....Boston, Mass.
Sherrerd, M. R....Newark, N. J.
Sherron, G. A....Norwalk, Conn.
Shertzer, T. B....New York City
Shuman, L. D...Philadelphia, Pa.
Shute, J. S.....New York City
Sikes, Z. H.....Yonkers, N. Y.
Simpson, J. T.....Newark, N. J.
Skillin, E. S....Glen Ridge, N. J.
Skinner, B. B....Brooklyn, N. Y.
Skinner, F. W.,
Tompkinsville, N. Y.
Skinner, J. F....Rochester, N. Y.
Sloan, S. A.....Philadelphia, Pa.
Small, J. H., Jr.Rutherford, N. J.
Smith, A. O..Port Jefferson, N. Y.
Smith, C. W....Westwood, N. J.
Smith, E. F.....Brooklyn, N. Y.
Smith, E. N.....Nutley, N. J.
Smith, G. W.,
Montreal, Que., Canada
Smith, H. S....Wilkes-Barre, Pa.
Smith, J. W.....New York City
Smith, W. F.....Valhalla, N. Y.
Smoley, C. K.....Scranton, Pa.
Smoyer, L. I.....New York City
Snell, T. C. B.....Newark, N. J.
Snow, C. H.....New York City
Snow, J. B....Woodhaven, N. Y.
Snyder, G. D.....New York City
Soest, H. C.....New York City
Solomon, G. R.....Atlanta, Ga.
Sonne, Otto.....New York City
Soper, G. A.....New York City
Spaulding, F. A..Pittsfield, Mass.
Spear, P. H.,
Hastings-on-Hudson, N. Y.
Spear, W. E.....New York City
Spencer, C. B....New York City
Spencer, Herbert..New York City
Spencer, T. N.,
West Philadelphia, Pa.
Splitstone, C. H..New York City
Sprague, E. L., Jr.,
White Plains, N. Y.
Stanton, W. D....Cohocton, N. Y.
Stark, C. W.....New York City
Starkweather, A. K..Nutley, N. J.
Starr, H. H.....New York City
Stearns, F. P.....Boston, Mass.
Stearns, R. H.....Boston, Mass.
Stehle, F. C.South Bethlehem, Pa.
Steinman, D. B..New York City
Stengel, C. H....New York City
Stephens, A. W..New York City
Stern, E. W.....New York City
Stevens, J. F....New York City
Stevenson, W. F.,
New Rochelle, N. Y.
Stiles, A. I.....New York City
Stoddard, G. C..New York City
Stone, W. W...Watertown, N. Y.
Storey, F. S.....New York City
Stowe, H. C.....Brooklyn, N. Y.
Stowitts, G. P....Yonkers, N. Y.
Strachan, Joseph..Brooklyn, N. Y.

Strachan, R. C.,
Richmond Hill, N. Y.
Strawn, T. C.....New York City
Strobel, C. L.....Chicago, Ill.
Strong, J. B.....Hillburn, N. Y.
Strong, M. R.....New York City
Stuart, J. C. K.,
Montreal, Que., Canada
Sutton, C. W.....Chester, Pa.
Sutton, Frank..Washington, D. C.
Swain, G. F.....Boston, Mass.
Sweeney, H. C....Brooklyn, N. Y.
Sweetland, H. A..Hartford, Conn.
Swensson, O. J....Yonkers, N. Y.
Swift, W. E.....Hartsdale, N. Y.
Swindells, J. S...Brooklyn, N. Y.

Taber, G. A.....Brooklyn, N. Y.
Taft, J. R.....New York City
Talbot, A. N.....Urbana, Ill.
Talbot, Earle.....Utica, N. Y.
Talbot, F. M.....New York City
Tallman, Leroy....Newark, N. J.
Tallman, P. B.East Orange, N. J.
Taylor, C. F.....New York City
Tenney, G. O.....Richmond, Va.
Thayer, N. A.....New York City
Thomas, C. D....Brooklyn, N. Y.
Thomas, W. J.....New York City
Thomes, E. H.....Jamaica, N. Y.
Thompson, J. A..New York City
Thompson, S. C...New York City
Thompson, S. E.,
Newton Highlands, Mass.
Thomson, S. F....Brooklyn, N. Y.
Thomson, T. K....New York City
Thorndike, S. H....Boston, Mass.
Throop, G. H..Mamaroneck, N. Y.
Thurber, C. D....Brooklyn, N. Y.
Tidd, A. W..White Plains, N. Y.
Tighe, J. L.....Holyoke, Mass.
Tilden, C. J.....Baltimore, Md.
Timberlake, S. M.,
East Orange, N. J.
Tolles, F. C.....Cincinnati, Ohio
Tompkins, E. DeV.New York City
Tooker, F. W..East Orange, N. J.
Torrey, J. E.....Paterson, N. J.
Tower, J. W.....New York City
Tozzer, A. C.....New York City
Trautwine, J. C., Jr.,
Philadelphia, Pa.
Travell, W. B...Ridgewood, N. J.
Travers-Ewell, A..New York City
Tretter, G. A.....Roanoke, Va.
Tribus, L. L.....New York City
Triest, W. G.....New York City
Trout, C. E.....New York City
Trow, F. H..Brown Station, N. Y.
Trowbridge, D. S..New York City
Tucker, L. W.....New York City
Tull, R. W.....New York City
Turner, D. L.....New York City
Turner, E. K.....Boston, Mass.
Turner, H. C.....New York City
Turner, J. P...Amsterdam, N. Y.
Tuska, G. R.....New York City
Tuttle, A. S.....New York City

Underhill, G. G....Newark, N. J.
Ungrich, M. J.....New York City
Upson, M. M.....New York City
Upton, Joseph....Flushing, N. Y.

Vail, E. M.....Plainfield, N. J.
Vallely, W. P.....New York City
Van Cleve, A. H..New York City
Van Horne, J. G..New York City
Van Keuren, C. A.,
Jersey City, N. J.
Van Norden, E. M.Brooklyn, N. Y.
Van Petten, A. A.,
Bayamon, Porto Rico
Verveer, E. L.....New York City
Vincent, E. F.....Denver, Colo.
Vincent, J. I.....New York City
Vredenburgh, Watson, Jr.,
Woodmere, N. Y.

Waddell, F. C.,
West New Brighton, N. Y.
Waddell, J. A. L.Kansas City, Mo.
Wadsworth, G. R.Brookline, Mass.
Wadsworth, J. E..New York City
Wagner, B. M.,
Rockville Center, N. Y.
Wagner, J. C....Philadelphia, Pa.
Wait, B. H...White Plains, N. Y.
Walker, C. I.....New York City
Walker, J. J..Dobbs Ferry, N. Y.
Waller, Percy....Rochester, N. Y.
Ward, C. D.....New York City
Ward, H. M.....Bloomfield, N. J.
Warnock, W. H...New York City
Wasser, T. J....Jersey City, N. J.
Watkins, F. W.,
White Plains, N. Y.
Watters, G. L.,
South Bethlehem, Pa.
Webster, G. S...Philadelphia, Pa.
Webster, M. A....Frankford, Pa.
Wegmann, Edward..Yonkers, N. Y.
Weitzner, H. M...New York City
Wells, C. E.....Yonkers, N. Y.
Wells, G. E....Naugatuck, Conn.
Welty, H. T.....Yonkers, N. Y.
Werbin, I. V.....New York City
Weymouth, Aubrey..Flushing, N.Y.
Wheatcroft, H. B..Newark, N. J.
Wheeler, W. S.....Lima, Peru
Wheelock, De F. A..Warren, Pa.
Whinery, Samuel,
East Orange, N. J.
Whiskeman, J. P..New York City
White, Lazarus....New York City
Whitney, C. S.....Ithaca, N. Y.
Whitney, F. O.....Boston, Mass.
Whitney, G. C....New York City
Whitson, A. U....Flushing, N. Y.
Whittemore, W. F.Hoboken, N. J.
Whittier, T. T....New York City
Wickham, H. R..Brooklyn, N. Y.
Wiggin, E. W.New Haven, Conn.
Wiggin, T. H....Scarsdale, N. Y.
Wilcock, Frederick,
Brooklyn, N. Y.
Wiley, W. H.....New York City
Williams, G. S..Ann Arbor, Mich.
Williams, J. P. J.,
Chautauqua, N. Y.
Willoughby, J. E.,
Wilmington, N. C.
Wilmot, James....New York City
Wilmot, Sydney...New York City
Wilson, C. A....Cincinnati, Ohio
Wilson, C. W. S.,
New Rochelle, N. Y.
Wilson, H. Z.....New York City
Wilson, T. L.....New York City
Wilson, T. W...Wilmington, Del.
Wilson, William..New York City
Wilson, W. T.....New York City
Winsor, F. E..White Plains, N. Y.
Winsor, G. A.....Valhalla, N. Y.
Wise, C. R.....Passaic, N. J.
Wise, R. S.....Passaic, N. J.
Witmer, F. P...East Orange, N. J.
Woehrlin, G. J....Brooklyn, N. Y.
Wolff, A. D., Jr.,
White Plains, N. Y.
Wood, G. P.....Peekskill, N. Y.
Woodruff, G. B.,
South Bethlehem, Pa.
Wyckoff, C. R...Hartsdale, N. Y.
Wyman, A. M.,
Long Island City, N. Y.

Yappan, Adolph.....Chicago, Ill.
Yates, B. C.....Lead, S. Dak.
Yates, E. A.....New York City
Yates, P. K.....New York City
Yates, W. H.....Albany, N. Y.
Yereance, W. B..New York City

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

March 3d, 1915.—8.30 P. M.—This will be a regular business meeting. A paper by Harold Perrine and George H. Strehan, Juniors, Am. Soc. C. E., entitled "Cinder Concrete Floor Construction Between Steel Beams", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

March 17th, 1915.—No paper is scheduled for this meeting, but a programme will be arranged and announced later.

ANNUAL CONVENTION

The Annual Convention of the Society will be held in San Francisco, Cal., September 16th, 17th, and 18th, 1915, being the Thursday, Friday, and Saturday immediately preceding the International Engineering Congress, which is to be held during the week beginning September 20th.

Three of the other National Engineering Societies, under whose auspices the International Engineering Congress is to be held, will also hold meetings in San Francisco at about that time.

Arrangements will be made for a special train, and possibly more than one train, to accommodate the members of all of these Societies who wish to attend their own meeting as well as the Congress.

This matter is in the hands of a Joint Committee of representatives of these organizations, and it is expected that within a short time a circular will be mailed to the entire membership of these Societies (some 28 000), giving in detail a schedule of train service, railroad and hotel rates, etc.

THE ENGINEERING FOUNDATION

On Wednesday, January 27th, 1915, at 8.30 P. M., a meeting to inaugurate the Engineering Foundation was held in the Auditorium of the United Engineering Societies Building, 29 West 39th Street, New York City, at which there was a large attendance. Mr. Gano Dunn, President of the United Engineering Society and Past-President of the American Institute of Electrical Engineers, presided, and announced that the Engineering Foundation had been started by a gift of $200 000 from Mr. Ambrose Swasey, of Cleveland, Ohio. Addresses were made by Dr. Henry Pritchett, President of the Carnegie Foundation for the Advancement of Teaching; Dr. Robert W. Hunt, Past-President of the American Institute of Mining Engineers; Mr. Charles Macdonald, Past-President of the American Society of Civil En-

gineers; and Dr. Alexander C. Humphreys, Past-President of the American Society of Mechanical Engineers.

Under this Foundation, as announced previously, this fund is to be administered by a Board consisting of two representatives from each of the four National Engineering Societies, the President of the United Engineering Society, *ex officio*, and two members to be elected at large.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

January 9th, 1915.—The meeting was held at the Albany Hotel; President Vincent in the chair; J. Y. Jewett, Vice-President, acting as Secretary; and present, also, 32 members and guests.

The minutes of the December, 1914, meeting were read and approved.

President Vincent announced that he would attend the meeting of the Presidents of the Local Associations to be held in New York City on January 19th, 1915.

On motion, duly seconded, a Committee on Legislation to represent the interests of the Association at the present session of the Colorado Legislature was appointed by the President, as follows: G. G. Anderson, Chairman, C. W. Comstock, J. E. Field, H. S. Crocker, and G. N. Houston.

On motion, duly seconded, a Committee to draft a bill on the Licensing of Civil Engineers for presentation to the Legislature, was appointed as follows: M. S. Ketchum, Chairman, J. E. Field, and Arthur Ridgway.

On motion, duly seconded, it was decided to call a special meeting of the Association for the discussion of the subject of licensing civil engineers, after the return of President Vincent from the New York meeting, and as a guide for the President, in discussing the subject before that meeting, a vote of the members present was taken with the following result: In favor, 10; against, 4; non-committal, 5; the remainder not voting.

A paper, on Unit Construction of Concrete Structures and a New Method of Placing Concrete Linings in Tunnels by Compressed Air, was presented by Mr. Shirley Houghton, assisted by Mr. C. S. Bumann, and illustrated with lantern slides. On account of the lateness of the hour, the paper was not discussed.

A vote of thanks was tendered Messrs. Houghton and Bumann for their interesting paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

(Abstract of Minutes of Meeting)

January 9th, 1915.—B. M. Hall in the chair; Thomas P. Branch, Secretary; and present, also, 6 members.

The minutes of the last meeting were read and approved.

The report of the Committee on the Revision of the Constitution was read by sections, amended, and adopted.

The report of the Committee on the Nominations of Officers was read and adopted, the following officers being elected: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. P. Branch.

The Chairman addressed the meeting in regard to the recognition of Juniors in some manner other than the mere printing of their names as such.

On motion, duly seconded, it was decided that the Association urge upon the Society: First, the institution of a student grade; second, some recognition of Juniors; and third, that the representative of the Association to the meeting of the Presidents of the Local Associations to be held in New York City on January 19th, 1915, be instructed to bring these matters before that meeting.

Adjourned.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

At its meeting of January 6th, 1915, the Board of Direction considered and approved the proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

The Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.) has been organized with the following officers: Frank M. Kerr, President; J. F. Coleman, and W. B. Gregory, Vice-Presidents; A. M. N. Blamphin, Treasurer; and L. C. Datz, Secretary.

Northwestern Association

At its meeting of November 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.). F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

At the meeting of the Association on October 5th, 1914, the following officers were elected for the ensuing year: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow and William Hunter;

Directors, John Sterling Deans, John W. Ledoux, Edgar Marburg, and H. S. Smith; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

(Abstract of Minutes of Meeting)

January 18th, 1915.—The meeting was called to order; President Richard L. Humphrey in the chair; W. L. Stevenson, Secretary; and present, also, 127 members and guests.

Mr. Edgar Marburg made a report on the activities of the Committee on Licensing Engineers. On motion, it was decided to recommend the appointment of a committee of the Society to investigate the subject of the licensing of engineers if the matter was not considered at the meeting of the Presidents of the Local Associations.

Letters by Messrs. Richard L. Humphrey and Charles Warren Hunt, relative to calling a meeting of the Presidents of the Local Associations, or their representatives, were read.

Mr. John Sterling Deans presented the resolution passed by the Board of Direction of the Association, relative to an Amendment to Article IV of its Constitution, providing that a certain number of Past-Presidents should be members of the Board, which was approved by the meeting.

President Humphrey announced some changes in the proposed charter of the "Engineers' Society of Philadelphia", and stated that the matter would be submitted in final form at the April meeting of the Association.

A letter from Mr. Charles Warren Hunt, Secretary of the Society, asking for suggestions as to the place of meeting of the Society, was read, and, on motion, the matter was referred to President Humphrey without instructions.

Charles D. Marx, President, Am. Soc. C. E., addressed the meeting on the Panama-Pacific Exposition and the Engineering Congress of 1915, and was tendered a rising vote of thanks for honoring the Association by being its guest and for his interesting address.

Adjourned.

Portland, Ore., Association

At the meeting of the Association on October 21st, 1914, the following officers were elected for the ensuing year: President, George C. Mason; First Vice-President, W. S. Turner; Second Vice-President, John T. Whistler; Treasurer, G. B. Hegardt; and Secretary, Charles J. McGonigle.

(Abstract of Minutes of Meeting)

January 8th, 1915.—The meeting was held at the Commercial Club; President Mason in the chair; Charles J. McGonigle, Secretary.

The meeting was held to discuss legislative matters of interest to engineers. The various laws affecting engineers and their work in Oregon were discussed by Messrs. Lewis, Mason, Newell, and others. On motion, duly seconded, it was decided that the Association go on record as opposed to a tax on water power.

It was moved, seconded, and carried that the Association favor placing under the office of the State Engineer, the investigation and ex-

ploitation of natural resources and the administration of all work of an engineering nature handled by the State.

On motion, duly seconded, it was decided that the Association request the United States Geological Survey to extend the topographical and river profile surveys of Snake River to Lewiston, Idaho.

On motion, duly seconded, it was ordered that the Legislative Committee consider the matter of the authority of City Engineers of cities in Oregon in laying out plats within 1 mile outside the city limits.

Adjourned.

St. Louis Association

At its meeting of October 7th, 1914, the Board of Direction considered and approved the proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

Seattle Association

(Abstract of Minutes of Meeting)

January 25th, 1915.—The Annual Meeting was called to order at 7.15 P. M.; President Ernest B. Hussey in the chair; Carl H. Reeves, Secretary; and present, also, 47 members and guests.

The President presented the guest of honor, the Rev. M. A. Matthews, who addressed the meeting, his subject being "Efficiency".

Mr. J. C. Ralston outlined the plan of the proposed Affiliated Engineering Society of Spokane and invited the members of the Association to attend the Third Annual Convention of that Society to be held in Spokane, on February 23d-25th, 1915.

President Hussey briefly outlined the work of the Association during 1914, and presented the report of Mr. John L. Hall, the representative from the Association at the meeting of the Presidents of the Local Associations, held in New York City on January 19th, 1915.

A unanimous vote of thanks was tendered Dr. Matthews for his helpful and instructive address.

The minutes of the meeting of December 28th, 1914, were read and approved.

The report of the Secretary-Treasurer for 1914 was presented and adopted.

The President read a letter from Charles Warren Hunt, Secretary of the Society, relative to holding a Spring meeting of the Society outside of New York City. On motion, duly seconded, the Association concurred in the plan for holding such meeting.

The officers for 1915 were nominated and elected, as follows: President, R. H. Ober; Vice-President, A. S. Downey; Secretary-Treasurer, Carl H. Reeves.

In reference to Mr. Hall's report of the meeting of the Presidents of the Local Associations, Mr. S. H. Hedges stated that, in his opinion, the Presidents of the proposed districts should be the Directors from those districts.

On motion, duly seconded, copies of Mr. Hall's report were ordered sent to the members of the Association, with the request that they make comments or offer suggestions for possible changes in the plan as outlined by the Majority Report of the Conference.

On motion, duly seconded, the Secretary-Treasurer was ordered to prepare a roster of the Association for 1915, and mail a copy to each member.

On motion, duly seconded, a vote of thanks was extended to the retiring officers in appreciation of the excellent work done by them.

Mr. Charles Evan Fowler addressed the meeting briefly on the duties and opportunities of engineers in assuming their share of work in public affairs.

Mr. Robert Howes, Chairman of the Conference Committee announced a meeting of the Joint Committee on the Federation of Technical Societies, to be held on January 27th, 1915.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 515 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers. Ulysses B. Hough is President.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Stresses in Railroad Track

March 18th, 1914.—The meeting was held in Chicago, Ill. Present, A. N. Talbot (Chairman), A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W.

G. Cushing, Emil Gerber, William McNab, G. J. Ray, F. E. Turneaure, and J. E. Willoughby.

The Committee met in joint session with the Committee on Stresses in Railroad Track of the American Railway Engineering Association, the two committees having decided to co-operate in the work. Of the Committee of the American Railway Engineering Association there were present W. M. Dawley, P. H. Dudley, H. E. Hale, J. B. Jenkins, and Paul M. LaBach.

There was a general discussion on the field of work and the way in which the work should be undertaken. Letters from men having special instruments and equipment for measuring deformations were read. A sub-committee consisting of A. N. Talbot, A. S. Baldwin, John Brunner, F. E. Turneaure, and W. M. Dawley was appointed to report on methods of organization and scope of work.

June 3d, 1914.—The meeting was held in Baltimore, Md. Present, A. N. Talbot (Chairman), John Brunner, Charles S. Churchill, C. G. E. Larsson, G. J. Ray, and J. E. Willoughby, of the Committee, and W. M. Dawley, P. H. Dudley, J. B. Jenkins, and Albert Reichmann. The report of the sub-committee on Methods of Organization and Scope of Work was discussed and adopted.

The following sub-committees were appointed:

Executive.—A. S. Baldwin, John Brunner, Charles S. Churchill, W. M. Dawley, and A. N. Talbot.

Tests.—John Brunner, W. J. Burton, W. M. Dawley, P. H. Dudley, H. E. Hale, J. B. Jenkins, Paul M. LaBach, C. G. E. Larsson, Albert Reichmann, A. N. Talbot, F. E. Turneaure, and J. E. Willoughby.

Ways and Means.—A. S. Baldwin, Charles S. Churchill, W. G. Cushing, W. M. Dawley, Robert W. Hunt, George W. Kittredge, William McNab, G. J. Ray, and Earl Stimson.

E. H. Fritch was chosen as Secretary of the Committee.

The programme of tests was left to the sub-committee on Tests, though the general idea was expressed that the first work should include the development of instruments and apparatus, and preliminary work on track, and that the latter should include the determination of the equilibrium depression curve of track as found statically with different weights of rail and different conditions of track, ballast, and roadbed. It was also decided that tests should be made on ballast and roadbed materials.

The general plan of carrying on the work was left to the Executive Committee, as was the matter of the budget and expenditures, and that Committee was asked to act as a financial committee. Other matters relating to the work of the Committee were discussed.

Special Committee on A National Water Law

January 20th, 1915.—The first regular meeting of the Committee was held at the House of the Society. Present, F. H. Newell (Chairman), Clemens Herschel, Robert E. Horton, Charles D. Marx, and Gardner S. Williams. The Chairman recited the various efforts made to hold a meeting of the Committee since its formation (June

4th, 1913), namely, at the last Annual Meeting, January 21st, 1914, and in Denver on April 9th, 1914. Not being able to meet a majority of the members at any one time, the Chairman had consulted individual members personally and by correspondence. As a result there had been prepared and mailed to each member of the Society a circular letter, dated June 30th, 1914, asking the co-operation of the members and for information. The replies received indicated general interest in the subject and also the fact that investments aggregating many millions of dollars are being delayed because of uncertainties growing out of lack of adequate State and National water laws; also that the fullest use and enjoyment of many natural resources are rendered impracticable by existing conditions which might be remedied through well-conceived legislation.

The conclusion reached was that the first part of the Resolution of June 4th, 1913, had been fulfilled, namely, that the Special Committee had investigated "the advisability of drafting a National Water Law applicable to all navigable, interstate and other waters within the jurisdiction of the United States, and embracing all uses of water," and that now the Committee deemed it advisable to enter upon the second part of the work authorized by the Resolution, namely, "to prepare a preliminary draft of such a law for submission at some regular meeting of the Society."

Inasmuch as the Committee approaches the subject from the standpoint of engineers rather than that of professional law makers, it was agreed that, before attempting to prepare a preliminary draft of a law, there should be agreement, as far as possible:

First, on the facts which necessitate such a law;
Second, on the principles which should underlie such a proposed law; and
Third, on the extent to which such principles may be embodied in a law.

Correspondence and replies to the circular letter have brought together many of the facts, but, in order that these may be more complete, it was agreed that additional correspondence should be entered into.

It was also understood that the individual members of the Committee should make suggestions as to the principles which should underlie such proposed law, and discuss, in sub-committees or otherwise, the more important of them.

The selection of a Secretary was deferred, the Chairman acting as such until more definite action could be taken.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building. Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From January 4th to February 1st, 1915)

DONATIONS*

AMERICAN SEWERAGE PRACTICE:

Volume II, Construction of Sewers. By Leonard Metcalf and Harrison P. Eddy, Members, Am. Soc. C. E. Cloth, 9½ x 6¼, illus., 10 + 564 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $4.00.

The first volume of this series on American Sewerage Practice was devoted to the Design of Sewers. In this, the second volume, the authors have taken up and described in detail the Construction of Sewers. Because such construction, to be of value, involves attention to many apparently unimportant details, the authors, it is stated, have discussed many things, in this volume, which usually are barely touched on in books and papers on sewerage. After giving in detail, for the student and engineer, the preliminary steps in sewer construction, such as maps, field work, examination of soil, borings, estimates, records, etc., the authors describe methods, machinery, and costs for excavation and tunneling, the purchasing, handling, and laying of various kinds of sewers, the contracts, specifications, and drawings for sewer work, and the operation and maintenance of sewers, as well as the causes and methods of prevention of explosions in sewers. The authors have had experience, it is stated, as designing and supervising engineers, not only on contract work, but also with day labor, of many miles of sewers, and they have endeavored in this book to present the instructions they would issue to superintendents and foremen in charge of such work and the advice they would give to contractors taking up sewer construction for the first time. It is hoped that the volume will prove useful to these men as well as to engineers and inspectors who should be familiar with practical methods of sewer construction. The Chapter headings are: Preliminary Investigations; Engineering Work and Inspection During Construction; Excavation; Machinery for Trench Excavation; Methods of Rock Excavation; Explosives and Blasting; Quantity and Cost of Excavation; Rate of Progress in Building Sewers; The Sheeting and Bracing of Trenches and Tunnels; Sizes of Sheeting, Rangers and Braces; Purchasing, Handling and Laying Sewer Pipe; Jointing Sewer Pipe; Construction of Brick and Block Sewers; Construction of Concrete Sewers; Profiles, Templates, Forms and Centers; Contracts, Specifications and Drawings; Technical Specifications; Operations and Maintenance of Sewerage Systems; Explosions in Sewers; Index.

MECHANISM OF STEAM ENGINES.

By Walter H. James and Myron W. Dole. Cloth, 9¼ x 6 in., illus., 10 + 170 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1914. $2.00.

The purpose of the authors, in this book, has been, it is stated, to present the subject in a manner that will make clear to the beginner the mechanical principles on which the steam engine operates, with special reference to the valve gear and governing devices, and the various diagrams used in the study of the engine. The book, it is stated, is intended as an elementary treatise on the kinematics of reciprocating engines and steam turbines, and is planned to meet primarily the needs of students who take up this subject as a part of, or immediately following, their course in the elements of mechanism and before they take up the study of the theory and practice of heat engineering or machine design. In order to illustrate the principles and methods described and to make them clearly understood, the authors have included examples, as well as detailed explanations, of the different types of mechanisms described. The Contents are: Introduction; General Discussion of a Reciprocating Steam Engine; Single-Valve Engines; Valve Diagrams; Typical Problems on the Slide-Valve Engine; Governing Devices for Single-Valve Engines; Riding Cut-Off Valves and Their Governing Devices; Multiple-Valve Engines; Hand-Operated Reversing and Controlling Gears; Valve Setting; Steam Turbines; Turbine Valve Mechanisms and Governors; Index.

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*Unless otherwise specified, books in this list have been donated by the publishers.

USE OF WATER IN IRRIGATION.

By Samuel Fortier. (Agricultural Engineering Series.) Cloth, 8¼ x 5¾ in., illus., 13 + 265 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $2.00.

This book, the author states, records mainly experiences in irrigation and irrigation methods gained in the field and laboratory. As here presented the subject-matter is said to deal with the agricultural side of irrigation, and the author's aim has been to benefit three classes of readers: the new settlers and those who are contemplating homes in the West; irrigation farmers and those interested in irrigation; and students in agricultural and engineering classes in high schools, colleges, and universities. The subject-matter, it is stated, deals almost exclusively with the irrigated farm and to problems that confront the irrigator, the legal, economic, and engineering phases being touched on only as they affect the welfare of the farmer. The author has described in detail methods of preparing the land, applying the water, and maintaining moisture conditions in the soil, and hopes that out of the methods described the farmer may adopt those best suited to his farm and thus pave the way for profitable returns. The Contents are: Introduction; The Irrigated Farm; The Necessary Equipment and Structures; Methods of Preparing Land and Applying Water; Waste, Measurement, Delivery and Duty of Water; Irrigation of Staple Crops; Index.

CONSTRUCTION OF MASONRY DAMS.

By Chester W. Smith, Assoc. M. Am. Soc. C. E. Cloth, 9½ x 6¼ in., illus., 11 + 279 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $3.00.

The preface states that this book has been written from the viewpoint of the constructing engineer, and the author hopes that it will supply details of construction and supervision which heretofore have not been adequately covered in treatises on this subject. As it has not been possible in a volume of this size to cover the whole subject of dams and dam construction, the author states that earthwork, rock excavation, pumping, cement, and source and distribution of power, have been omitted or are only touched on in the most elementary manner. In Chapter XIV is described the estimating of costs, and cost figures are given which it is hoped will prove reliable and useful guides for construction estimates. There is also included a partial list of existing dams with descriptions and costs. The Contents are: Exploring the Site; Temporary Works for Stream Diversion; Preparing the Foundation; Masonry Construction; Quarrying; Face Work; Miscellaneous Features; Plant and Power; Installation Required and Power Consumption; Assembling Materials, Crushing and Mixing; Transportation of Materials; Probable Future Methods; Estimates of Cost; Partial List of Existing Dams with Description and Costs; Index.

Gifts have also been received from the following:

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Breithaupt, W. H. 1 bound vol.
Buffalo, N. Y.-Bureau of Water. 1 vol.
Buffalo, N. Y.-Dept. of Public Works. 1 vol.
Bureau of Ry. Economics. 3 pam.
California, Univ. of. 1 vol.
Canada-Dept. of Marine and Fisheries. 1 vol., 1 pam.
Canada-Water Power Branch. 4 vol., maps.
Canadian Northern Ry. Co. 1 pam.
Case School of Applied Science. 1 vol.
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New York City Record. 1 bound vol.
Niagara Falls, N. Y.-City Clerk. 1 pam.
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Richmond, Fredericksburg & Potomac R. R. Co. 1 pam.
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Ross, D. W. 1 pam.
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U. S.-Forest Service. 1 pam.
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Universidad Nacional de la Plata. 1 pam.
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Washington, Univ. of. 1 pam.
Western Canada Irrig. Assoc. 2 pam.
Western New York & Pennsylvania Ry. Co. 6 pam.
Wisconsin, Univ. of. 1 pam.
Withington, Sydney. 1 pam.
Wyoming-State Engr. 1 bound vol.
Yale Univ. 1 pam.

BY PURCHASE

Die Glas- und Wellblechdeckung der Eisernen Dächer. Von Th. Landsberg. Darmstadt, 1887.

A Text-Book of Mechanical Engineering. By Wilfrid J. Lineham. Part 1, Workshop Practice; Part 2, Theory and Examples. Eleventh Edition, Revised. London, 1914.

Carrying Out the City Plan: The Practical Application of American Law in the Execution of City Plans. By Flavel Shurtleff, in Collaboration with Frederick Law Olmsted. New York, 1914.

Vorlesungen über Wasserkraftmaschinen. Von R. Camerer. Leipzig und Berlin, 1914.

Proceedings of the Seventeenth Annual Meeting of the American Society for Testing Materials; Vol. 14, Part 2. Edited by the Secretary. Philadelphia, 1914.

Transactions of the Institution of Mining and Metallurgy: Twenty-second Session; 1912-13; Vol. 22. New York and London, 1913.

Cranes, Their Construction, Mechanical Equipment and Working. By Anton Böttcher. Translated and Supplemented with English, American, and Continental Practice, by A. Tolhausen. London, 1908.

Iron Ores: Their Occurrence, Valuation and Control. By Edwin C. Eckel. New York and London, 1914.

The Autocar Imperial Year Book, 1914. London.

Das Spiel der Kräfte im Verbundbalken: Eine Darstellung der Lehre vom Verbund, von der Spannungsverteilung und Ermittlung in auf Biegung Beanspruchten Eisenbetonkörpern. Von Georg Fischer. Lissa, I. P., 1914.

Taschenbuch für Bauingenieure. Von Max Foerster. Zweite, verbesserte, und erweiterte Auflage. Berlin, 1914.

Rural Improvement: The Principles of Civic Art Applied to Rural Conditions, Including Village Improvement and the Betterment of the Open Country. By Frank A. Waugh. New York and London, 1914.

A Popular Treatise on the Colloids in the Industrial Arts. By Kurt Arndt. Translated from the Second Enlarged German Edition, by Nahum E. Katz. Easton, Pa., and London, 1914.

The Deposits of the Useful Minerals and Rocks: Their Origin, Form, and Contents. By F. Beyschlag, J. H. L. Vogt, and P. Krusch. Translated by S. J. Truscott. Vol. 1. London, 1914.

Principes et Exemples d'Architecture Ferronniere les Grandes Constructions Edilitaires en Fer la Halle-Basilique. Par L. A. Boileau. Paris, 1881.

The Engineering Index Annual for 1914: Compiled from the Engineering Index Published Monthly in the *Engineering Magazine* During 1914. New York, 1915.

Installing Efficiency Methods. By C. E. Knoeppel. New York, 1915.

SUMMARY OF ACCESSIONS

(From January 4th to February 1st, 1915)

Donations (including 15 duplicates)	249
By purchase	17
Total	266

MEMBERSHIP

(From January 8th to February 4th, 1915)

ADDITIONS

MEMBERS

		Date of Membership.
Cooper, David Reginald. Hydr. Engr., 500 Isham St., New York City	Assoc. M.	July 1, 1908
	M.	Dec. 2, 1914
Corning, Dudley Tibbits. 30 Belle Ave., Troy, N. Y.	Assoc. M.	Oct. 1, 1902
	M.	Jan. 6, 1915
Mitchell, Louis Adolph. Supt. of Roadway and Bldgs., Union Fraction Co. of Indiana, Anderson, Ind.	Assoc. M.	May 31, 1910
	M.	Jan. 6, 1915
Powelson, Wilfrid Van Nest. Cons. Engr., 60 Wall St., New York City		Jan. 6, 1915
Rogers, Clarence Mord. Engr., Board of Public Works, Daytona, Fla.		Jan. 6, 1915
Rolfe, William Edward. Associate to Pres., Board of Public Impvts., 301 City Hall, St. Louis, Mo.		Jan. 6, 1915
Ross, Thomas Alexander. Chf. Engr., Magadi Ry., Magadi Junction, *via* Mombasa, British East Africa	Assoc. M.	Dec. 3, 1902
	M.	Oct. 7, 1914
Stengel, Carl Henry. Chf. Engr., Eng. and Constr. Dept., United Realty & Impvt. Co., 111 Broadway (Res., 2674 Jerome Ave., Fordham), New York City	Assoc. M.	Oct. 3, 1906
	M.	Jan. 6, 1915
Thorndike, Sturgis Hooper. Cons. Engr. (Fay, Spofford & Thorndike), 308 Boylston St., Boston, Mass.		Jan. 6, 1915
Waldron, Albert Edwin. Maj., Corps of Engrs., U. S. A., Washington Barracks, Washington, D. C.	Assoc. M.	July 1, 1909
	M.	Jan. 6, 1915

ASSOCIATE MEMBERS

Aldrich, Elbert Christopher. City Engr., Auburn, N. Y.		Jan. 6, 1915
Beck, Alfred Fortin. Chf. Designer and Draftsman, Bridge Dept., Seaboard A. L. Ry., 712 Duke St., Norfolk, Va.		Jan. 6, 1915
Blight, Arthur Frederick. Asst. Hydr. Engr., Pacific Light & Power Corporation, 603 Pacific Elec. Bldg., Los Angeles, Cal.	Jun.	Feb. 28, 1911
	Assoc. M.	Jan. 6, 1915
Carew, Frank Jerome. Care, Herbert Howe, 1040 Faile St., New York City		Jan. 6, 1915
Chivers, Norman Moore. Junior Engr., U. S. Engr. Dept., Elizabethtown, N. C.		Jan. 6, 1915
Collins, Richard Vincent. Res. Engr., State Highway Dept., Hornell, N. Y.		Jan. 6, 1915

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
DAVY, JESSE JOHN. Dist. Engr., State Highway Comm., Preston, Minn		Oct. 1, 1912
DUBUIS, JOHN. Eng. Insp., Desert Land Board, State of Oregon, 1155 North Summer St., Salem, Ore	Jun.	Oct. 1, 1912
	Assoc. M.	Jan. 6, 1915
EGE, CHARLES RAYMOND. Prin. Asst. Engr. for State Highway Commr., P. O. Box 14, Olympia, Wash		Jan. 6, 1915
ELLIOTT, JOHN ARTHUR. Dist. Engr., Oregon State Highway Comm., Wyeth, Ore		Jan. 6, 1915
HALL, CHARLES LACEY. First Lieut., Corps of Engrs., U. S. A., 305 Southern Bldg., Washington, D. C	Jun.	June 30, 1910
	Assoc. M.	Jan. 6, 1915
HARRIS, FRANK SAMPSON MASON. Asst. Engr., Panama-Pacific International Exposition, 454 Sixty-first St., Oakland, Cal	Jun.	Mar. 5, 1912
	Assoc. M.	Jan. 6, 1915
HAYES, FERDINAND EUGENE, JR. Care, Bethlehem Chili Iron Mines Co., La Higuera, Coquimbo, Chili	Jun.	Feb. 4, 1908
	Assoc. M.	Nov. 4, 1914
HOWSON, GEORGE WILLIAM. Civ. and Hydr. Engr., Sierra & San Francisco Power Co., 58 Sutter St., San Francisco, Cal.	Jun.	Oct. 5, 1909
	Assoc. M.	Jan. 6, 1915
HOWSON, LOUIS RICHARD. Prin. Asst. Engr., Alvord & Burdick, 641 East 50th Pl., Chicago, Ill		Dec. 2, 1914
JOHNSTON, WALTER WIER. 1300 Leighton Ave., Anniston, Ala		Jan. 6, 1915
KENNISON, KARL RAYMOND. Prin. Asst. to John R. Freeman, 815 Grosvenor Bldg., Providence, R. I		Jan. 6, 1915
LINDERS, JOHN EDWARD AUGUST. Engr. in Chg., Drawing Room, The J. W. Frazier Co., 2205 Bellefield Ave., Cleveland, Ohio		Sept. 2, 1914
MASURY, ALFRED FELLOWS. Chf. Engr., International Motor Co., 345 West 70th St., New York City		Jan. 6, 1915
MUMFORD, GURDON SALTONSTALL. Pres., Unit Wall Constr. Co., 1328 Broadway, New York City		Sept. 2, 1914
PORTER, ELMER ALFRED. Dist. Engr., U. S. Geological Survey, 421 Federal Bldg., Salt Lake City, Utah	Jun.	Oct. 1, 1912
	Assoc. M.	Jan. 6, 1915
REDDICK, JOHN BURKE. Civ., Hydr. and Min. Engr. (Jones, Hanscom & Reddick, Inc.), 102 Humboldt Bank Bldg., San Francisco (Res., 3014 College Ave., Berkeley), Cal		Sept. 2, 1914
RUGE, BERNARD ARNOLD. Secy.-Treas., Ward Carpenter & Co., Inc., 95 Franklin St., Tarrytown, N. Y		Jan. 6, 1915
SADLER, CHARLES BOONE. Asst. Engr., Baltimore Sewerage Comm., 907 Arlington Ave., Baltimore, Md		Jan. 6, 1915

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
SAWYER, ERNEST WALKER. Canadian Mgr., The Universal Radio Syndicate, Ltd., Newcastle, N. B., Canada	Jun.	May 7, 1913
	Assoc. M.	Jan. 6, 1915
SICKLES, RAYMOND. Engr., Larkin & Sangster, Seneca Falls, N. Y.		Sept. 2, 1914
TIFFANY, NATHAN NEWTON. Civ. Engr. and Surv.; Member, Board of Supervisors, Suffolk County, East Hampton, N. Y.	Jun.	June 5, 1906
	Assoc. M.	Jan. 6, 1915
TITUS, WILLIAM JACY. 1710 Central Ave., Indianapolis, Ind.		Dec. 2, 1914
WAALKES, GERRIT. Chf. Draftsman, Mt. Vernon Bridge Co., 201 North Park St., Mt. Vernon, Ohio		Jan. 6, 1915
WHITNEY, FRANKLIN LEMUEL. Care, Honduras Drug Co., La Ceiba, Honduras		Nov. 4, 1914
YOUNG, CLARENCE RICHARD. Asst. Prof. of Structural Eng., Univ. of Toronto, 98 Hilton Ave., Toronto, Ont., Canada		Jan. 6, 1915

ASSOCIATES

BRITTON, JOHN ALEXANDER. Vice-Pres., Gen. Mgr., and Chf. Engr., Pacific Gas & Elec. Co., 445 Sutter St., San Francisco, Cal.	Jan. 6, 1915
MICKEY, CLARK EDWIN. Asst. Prof. of Applied Mechanics, Univ. of Nebraska; Cons. and Testing Engr., Lincoln, Nebr.	Jan. 6, 1915

JUNIORS

ANDERSON, JOHN HENNING. Road Advisor, Universal Portland Cement Co., 208 South La Salle St., Room 1532, Chicago, Ill.	Jan. 6, 1915
BICKERTON, WILBUR EARL. In Chg., Designing and Estimating Dept., Trussed Concrete Steel Co., 141 Milk St., Boston, Mass.	Jan. 6, 1915
CONNELLY, WALTER LOUIS. Recorder, Detroit Railway Comm., 601 Genesee St., Saginaw, Mich.	Jan. 6, 1915
DAVIS, WILLIAM EILERT. Box 404, Ventura, Cal.	Nov. 4, 1914
DURANT, WILLIAM BULLARD. Asst. Engr., Turners Falls Co., Turners Falls, Mass.	Jan. 6, 1915
FISKE, HAROLD LASELLE. Engr., The Foundation Co., 9 McClellan St., Schenectady, N. Y.	Jan. 6, 1915
FONT, MANUEL. Asst. San. Engr., Service of Sanitation, San Juan, Porto Rico	Jan. 6, 1915
LOWERY, JOHN, JR. 541 Maplewood Ave., Ambridge, Pa.	Jan. 6, 1915
MOONE, MELVIN LYMAN. Draftsman, State Highway Dept., Lansing, Mich.	Jan. 6, 1915

JUNIORS (*Continued*)

	Date of Membership.
OSBORN, JESSE THOMAS. Asst. Engr., C. M. & St. P. Ry., 1359 Ry. Exchange Bldg., Chicago, Ill	Jan. 6, 1915
QUINCY, EDMUND. Care, Eng. Dept., Q. & C. Co., 90 West St., New York City	Jan. 6, 1915
STANFORD, JAMES LELAND. 217 Furnald Hall, Columbia Univ., New York City	Jan. 6, 1915
TAUSSIG, JOHN WRIGHT. Care, Raymond Concrete Pile Co., 140 Cedar St., New York City	Jan. 6, 1915
WEBSTER, MAURICE ANDERSON. 4900 Penn St., Frankford, Philadelphia, Pa	Sept. 2, 1914

REINSTATEMENTS

MEMBER

	Date of Reinstatement.
PHILLIPS, RICHARD HARVEY	Jan. 30, 1915

RESIGNATIONS

ASSOCIATE MEMBER

	Date of Resignation.
PAIGE, JASON	Jan. 6, 1915

DEATHS

JACOMB-HOOD, JOHN WYKEHAM. Elected Member, February 5th, 1902; died March 6th, 1914.

KEEFER, THOMAS COLTRIN. Elected Member, April 4th, 1877; Honorary Member, October 1st, 1913; died January 7th, 1915.

RASINSKY, CHARLES EZEKIEL. Elected Associate Member, July 9th, 1912; died December 26th, 1914.

Total Membership of the Society, February 4th, 1915, 7 704.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(January 2d to February 1st, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
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(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Steel and Iron,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 20c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.

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The Transportation Problem on Surveys in the Canadian North. J. A. MacDonald. (96) Jan. 21.
Feldspar as a Possible Source of American Potash. Allerton S. Cushman and George W. Coggeshall. (Paper read before the Am. Inst. of Chemical Engrs.) (105) Feb.
Increased Taxation of Public Utilities. Edward Gruhl. (Paper read before the Wisconsin Elec. Assoc.) (83) Feb. 1.
Bautechnische Verwertung des Torfes, insbesondere zur Schalldämpfung.* Guntram Mahir. (7) Oct. 17.

Municipal.

Proposed Standard Specifications for One and Two Course Concrete Highway.* (110) Oct.
Proposed Standard Specifications for One and Two Course Concrete Street Pavement. (110) Oct.
Road Improvement. W. A. McLean. (Paper read before the Am. Road Builders' Assoc.) (96) Dec. 31.
Brick Roads and Streets. John Laylin. (60) Jan.
Concrete Curb and Gutter as Constructed in Denver, Colo.* E. B. Van de Greyn. (60) Jan.
Concrete for Paving.* G. W. Pickels. (60) Jan.
Convict Labor on Country Roads.* George C. Warren. (Paper read before the General Federation of Women's Clubs.) (60) Jan.
Economic Limit of Pavement Repairs. George H. Norton. (36) Jan.
Traffic Congestion Problem in Brooklyn, N. Y. W. H. Messenger. (Paper read before the Brooklyn Engrs.' Club.) (60) Jan.
Better Brick Roads for Illinois.* Charles D. Stillwell. (From the *Farmers' Review.*) (76) Jan. 5.
Conditions Determining Maximum Grades and Methods and Cost of Road Grading in West Virginia.* A. D. Williams. (Paper read before the Am. Road Congress.) (86) Jan. 6.
Road Location and the Economics of Road Improvement. D. Tucker Brown. (86) Jan. 6.
Wood Block Pavement After 6 Years Service.* (14) Jan. 9.
Wood Block Pavement without Cushion Layer of Sand, Experimental Section in Business Center of Chicago, Laid Directly on Concrete Base and Covered with Hot Pitch and Sand.* (14) Jan. 9.
Analyses of Asphaltic Concrete and Asphalt Block Laid in Washington, D. C., in 1914. (From Report of the Engineer Department.) (86) Jan. 13.
The Cementing Value of Bituminous Binders and Results Obtained in Some Commercial Tests.* Lester Kirschbaum. (From *Journal of Industrial and Engineering Chemistry.*) (86) Jan. 13.
Standard Concrete Culverts Recommended by the Michigan Highway Department. (86) Jan. 13.
The Columbia Highway in Oregon.* (13) Jan. 14.
Notes on Concrete Road Construction. (Abstract of paper read before the Good Roads Congress.) (13) Jan. 14.
Protecting Newly Paved Streets. (13) Jan. 14.
Resurfacing Asphalt Pavements in San Francisco, Cal. James Owen. (13) Jan. 14.
Subways for Public-Utility Pipes and Wires in Chicago Streets. (13) Jan. 14.
Destructor Clinker Pavements. George Mawbey. (104) Jan. 15.
Bituminous Macadam by the Cold-Mixing Method. (14) Jan. 16.
Test Pavement of Creosoted Blocks at Kansas City. (14) Jan. 16.
Analysis of Concrete Curb Construction for Suburban Improvements near New York City.* Allen C. Haskell. (86) Jan. 20.
A City Street Traffic Investigation; Bridgeport, Conn.* Alfred F. Muller. (13) Jan. 21.
Electric Street Sprinkler does Work of Three Teams.* (14) Jan. 23.
Costs Reduced by Monolithic Curb, Gutter and Pavement.* (14) Jan. 23.
Pavement Troubles Along Car Tracks Eliminated; Wood Block Laid on 1:4 Cement and Sand Cushion Sprinkled with Water, Prove Satisfactory at Newark, N. J. (14) Jan. 23.
An Example of the Improvement of a Section of Lightly Travelled Country Road Using Funds Limited in Amount. S. P. Hooker. (Paper read before the Am. Road Congress.) (86) Jan. 27.
Power Road Machinery with Special Reference to Hauling and Earth Road Grading.* N. DeWind. (Paper read before the Missouri State Engrs.' Assoc.) (86) Jan. 27.

*Illustrated.

Municipal—(Continued).

Analysis of Cost of Hauling on Country Roads.* T. R. Agg. (Abstract of paper read before the Good Roads Congress.) (13) Jan. 28.
Brick Pavement Experience in Toronto, Ont.* (13) Jan. 28.
Ueber die Art der Strassenbenennung unter besonderer Berücksichtigung Gross-Berliner Verhältnisse. E. Maul. (39) Nov. 20.

Railroads.

The Illinois Central New Central Station and Track Elevation, Memphis, Tenn.* (87) Oct.; (15) Jan. 29.
Proper Method of Rail Renewals. A. M. Clough. (Paper read before the Roadmasters' and Maintenance of Way Assoc. of Am.) (87) Oct.
Proper Type of Track Accessories. M. Donahoe. (Paper read before the Roadmasters' and Maintenance of Way Assoc. of Am.) (87) Oct.
Single Track Automatic Block Signals Between Sioux City and Missouri Valley, Chicago & North Western.* (87) Oct.
Gray-Thurber System of Automatic Train Control as Applied to an Automatic Signal System.* (87) Nov.
Reinforced Concrete Fence Posts, B. R. & P. Ry.* (87) Nov.
Installation of Twenty-First Street Crossings, Chicago.* A. M. Cornell. (87) Nov.
Method and Progress of Lining C., B. & Q. Ry. Tunnel at Arminto, Wyo.* (87) Dec.
The Parcel Post and Its Effect on Railway Revenues. V. J. Bradley. (65) Dec.
Railroad Valuation. H. H. Edgerton. (87) Dec.
Signals and Interlocking, Jersey City Terminals, Central Railroad of New Jersey.* (87) Dec.
Standard Mileage and Sea Level Datum for Railways. (87) Dec.
The Selection of a Railroad Track Scale.* Eugene Motchman. (62) Dec. 21.
Automatic Train Stops; Chicago and Eastern Illinois R. R.* (21) Jan.
Axle-Boxes 0-6-2 Tank Engines; Taff Vale Railway.* (21) Jan.
Calculation of Values in A. C. Track Circuiting.* (21) Jan.
Electrical Resonance Apparatus for Track-Circuiting Purposes.* (21) Jan.
Erie Caboose with Steel Center Sills.* (25) Jan.
The Grinding Wheel in the Railroad Shop.* B. F. Jacobs. (108) Jan.
Lubrication of Car Journals. W. A. Clark. (25) Jan.
Riveting in Steel Car Construction.* H. A. Hatfield. (25) Serial beginning Jan.
Sanitary Drinking Fountain for Passenger Cars.* (15) Jan.
2-10-0 Simple Expansion and Compound Locomotives, Swiss Federal Railways.* (21) Jan.
Steam Locomotives of To-day. (Report of Sub-Committee, Am. Soc. of Mech. Engrs.) (55) Jan.
Steel Coaches for the Santa Fé.* (25) Jan.
A Study of Spring Rigging Design.* J. P. Shamberger. (25) Jan.
1 500 Horse Power Locomotive for the Midi Railway.* (11) Serial beginning Jan. 1.
Baggage-Buffet Cars for the Union Pacific R. R.* (18) Jan. 2.
Coaling and Sanding Station on the Virginian Railway at Elmore.* F. F. Harrington. (14) Jan. 2.
Developments with Steel Ties on the Bessemer & Lake Erie R. R.* (18) Jan. 2.
Electrification of the Puget Sound Lines of the Chicago, Milwaukee & St. Paul Railway.* (111) Jan. 2; (13) Jan. 7; (18) Jan. 16; (19) Jan. 23.
New Locomotive Repair Shops of the Chicago & Alton R. R. at Bloomington, Ill.* (18) Serial beginning Jan. 2; (15) Jan. 8.
The Salt Lake & Utah Railroad.* (17) Jan. 2.
Lining Long Tunnels and Tunnels Subjected to Heavy or Eccentric Ground Pressure.* E. Lauchli. (96) Jan. 7.
A Simple Form for Earthwork Computations.* E. F. Verplanck. (13) Jan. 7.
Beam-Light Signals on the Pennsylvania.* (15) Jan. 8.
Combination Through Classification and Terminal Yard. W. C. Copley. (15) Jan. 8.
Earthworks.* G. Bertram Hartfree. (104) Jan. 8.
Extension of the Bakerloo Tube.* (12) Serial beginning Jan. 8.
Mechanical Stokers on American Locomotives. (12) Serial beginning Jan. 8.
The Simple Train Staff.* (15) Jan. 8.
British-Built Four-Cylinder Express Locomotive for the São Paulo Ry., Brazil.* F. C. Coleman. (18) Jan. 9.
Electrical Operation of the B. A. & P. Ry.* J. B. Cox. (82) Jan. 9.
Motor Trucks and Trailers in Freight Houses. (14) Jan. 9.
Rolled and Forged Steel Pistons.* W. W. Scott, Jr. (Paper read before the Ry. Club of Pittsburgh.) (18) Jan. 9.
Electric Train Staff on the Canadian Pacific.* E. S. Taylor. (15) Jan. 15.
Gasolene Switching Locomotive.* (15) Jan. 15.
Locomotive Tests on the Chicago & North Western.* (15) Jan. 15.

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Railroads—(Continued).

St. Paul and Oregon-Washington Joint Terminals in Spokane; Extensive Grade Separation Work Required to Reach Passenger and Freight Stations in Heart of City.* (15) Jan. 15.
Smoke Exhauster for the St. Paul Engine House at Chicago.* W. S. Lacher. (15) Jan. 15.
Tank Locomotive for the Great Central Railway.* (11) Serial beginning Jan. 15.
The Alaskan Railroad Surveys.* D. L. Reaburn. (13) Jan. 21.
An Automatic Cab-Signal System for Stopping Trains.* (13) Jan. 21.
Burned-Clay Ballast. (13) Jan. 21.
Canadian Railway Development in 1914. (96) Jan. 21.
Interlocking at North Philadelphia.* W. M. Post. (15) Jan. 22.
Pacific and Mikado Type Locomotives for the New Orleans & Northeastern.* (15) Jan. 22.
Construction of the Hallstead Cut-Off, D., L. & W. R. R.* (18) Serial beginning Jan. 23.
Gunite Concrete Encasement, Chicago, Rock Island & Pacific Ry.* (18) Jan. 23.
Highway-Crossing Protection.* (17) Jan. 23.
Justifiable Expenditure for Tie Treatment. Harrington Emerson and T. T. Bower. (Abstract of paper read before the Am. Wood Preservers' Assoc.) (14) Jan. 23.
Mechanical Life of Ties as Affected by Ballast. Earl Stimson. (Abstract of paper read before the Am. Wood Preservers' Assoc.) (14) Jan. 23; (18) Jan. 23.
Sill Ties, Treated or Untreated Timber or Concrete? F. J. Angier. (Abstract of paper read before the Am. Wood Preservers' Assoc.) (14) Jan. 23.
Steel Tie Construction in Electrically Warmed Concrete.* Julian M. Bamberger. (17) Jan. 23.
A Comparison of the Old and New Lines of the Canadian Pacific at Rogers Pass, B. C. (15) Jan. 29.
Fuel Economy on the Great Western.* (15) Jan. 29.
Standardization of Methods in the Railroad Shop. Ernest Cordeal. (9) Serial beginning Feb.
La Nouvelle Gare Badoise, à Bale (Suisse).* (33) Dec. 15.
Installation pour le Lavage à Chaud des Locomotives et le Chauffage du Dépot de Machines de la Gare de Turin.* P. Calfas. (33) Jan. 16.
Le Projet de Raccordement entre les Gares de Cornavin et des Eaux-Vives à Genève.* (33) Jan. 16.
Stützmauern aus Eisenbeton.* H. W. K. Ziesemer. (80) Serial beginning Oct. 8.
Die Tragkraft des Zusammenhaltes der Erde.* A. Francke. (102) Serial beginning Oct. 15.
Die Hartholzverdübelung.* E. Biedermann. (102) Serial beginning Oct. 15.
I C + C. IV. T. F. G.—Locomotive der südafrikanischen Eisenbahnen.* (102) Oct. 15.
Schienenstühle auf kiefernen Schwellen.* E. C. W. van Dyk. (102) Oct. 15.
Eisenbeton im Eisenbahnoberbau.* (80) Oct. 31.
Ein neues Verfahren zur Reinigung der Heiz- und Rauch-Rohre von Kesselstein und Flugasche. Buckart. (102) Nov. 15.
Der Saalwagen Nr. 510 der österreichischen Staatsbahnen.* J. Fleischmann. (102) Nov. 15.
Formänderungen am schwebenden Schienenstosse.* H. Saller. (102) Nov. 15.
Die natürliche Böschung von Erdarten starken Zusammenhaltes.* A. Francke. (102) Nov. 15.
Fortschritte im Baue von Verschiebewinden.* Wintermeyer. (102) Nov. 15.
Die Furkabahn.* (107) Dec. 19.
Annahme hochgespannten Gleichstroms für die Elektrisierung der Chicago, Milwaukee und St. Paul-Eisenbahn.* Zehme. (41) Jan. 7.

Railroads, Street.

The Safety First Movement: How Electric Railways Have Worked to Reduce Accidents.* (17) Jan. 2.
Inclosed Prepayment Cars for Baltimore.* (17) Jan. 9.
A Two-Mile Street Railway Tunnel at San Francisco; City Undertakes to Build Tunnel under Twin Peaks. (14) Jan. 9.
Brooklyn Bridge Subway Connection, New York City.* (14) Jan. 16.
The Double Trolley System in Seattle.* H. J. Kennedy. (17) Jan. 16.
Rapid Steel Erection on Third Track Work, Second Avenue Elevated, New York.* (14) Jan. 16.
Cleveland Railway's New Repair Shops.* (17) Jan. 23.
Flange-Bearing Track Intersections for Street Railways.* (13) Jan. 28.
Fireproof Carhouse at Vancouver, B. C.* (17) Jan. 30.
Versenkung eines hölzernen Kastenfangedammes beim Bau des Spreetunnels der Berliner Untergrundbahn.* Benno Stecher. (40) Oct. 10.
Die Kabelführung städtischer Schnellbahnen.* (41) Nov. 12.
Die Erweiterungen des Schnellbahnnetzes von Gross-New York.* Brugsch. (51) Serial beginning Jan. 6.

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Sanitation.

Gas-Heated Hot-Water Apparatus.* T. F. C. Patterson. (Paper read before the London and Southern District Junior Gas Assoc.) (66) Dec. 22.
The Purification of Dairy Wastes. G. Bertram Kershaw. (104) Serial beginning Dec. 25.
Design, Maintenance, and Operation of Plants for Treatment of Sewage. (Report of Committee of the Am. Public Health Assoc.) (96) Dec. 31.
A Method of Testing the Efficiency of Distribution of Sewage Sprinkler Nozzles.* Edward R. Stapley. (36) Jan.
Street Cleaning in New York. Frederick L. Stearns. (Paper read before the Municipal Engrs. of the City of New York.) (60) Jan.
Excavating and Backfilling Sewer Trenches by Machine; Detailed Costs and Explanation of Method used in Narrow Lanes and Business Streets in Vancouver, B. C. (14) Jan. 2.
Recommended Tests for Use in Laboratory Control of Sewage Works Operation. (Report of the Am. Public Health Assoc.) (86) Jan. 6; (104) Jan. 1; (13) Jan. 14.
Buffalo Sewage Disposal and Water-Supply in Relation to the Pollution of the Great Lakes. (13) Jan. 7.
Church Warmed by Cast-Iron Sectional Boiler.* (101) Jan. 8.
Steam Disinfection for Sewage on Common Carriers; U. S. Public Health Service has Devised Apparatus for Use on Railway Coaches and Lake Vessels; Costs of Construction and Operation. (14) Jan. 9.
Heating, Ventilation, and Refrigeration. (Report of Committee of the Am. Commercial Gas Assoc.) (66) Jan. 12.
An Electric Sewage-Pumping Station, Audubon, N. J.* (13) Jan. 14.
Sewage Purification at the Experimental Station of the Ontario Board of Public Health.* (From *Bulletin No. 2*, Ontario Health Board.) (96) Jan. 14.
Multiple Flowing-Through Chambers in Imhoff Tanks.* (13) Jan. 16.
Ozone in Ventilation. J. C. Olsen and Wm. H. Ulrich. (Paper read before the Am. Inst. of Chemical Engrs.) (19) Jan. 16.
A Privately Financed System of Sewers. Henry W. Taylor. (14) Jan. 16.
Summer Municipal Wastes. (14) Jan. 16.
Baltimore Sewage Pumping Plant.* Warren O. Rogers. (64) Jan. 19.
The Prevention of Odors at City Refuse Disposal Works. Rudolph Hering. (Paper read before the Am. Public Health Assoc.) (86) Jan. 27.
An Ash and Refuse Dumping Board, West 77th St., North River, New York City.* C. W. Staniford. (13) Jan. 28.
Sewage-Treatment and Garbage-Reduction Works for Akron, Ohio. (14) Jan. 28.
Designing of Warm Air Heating Systems.* Roy E. Lynd. (Paper read before the Soc. of Heating and Ventilating Engrs.) (101) Jan. 29.
New Methods of Pneumatic Tunneling Aid Safe and Rapid Completion of Passaic Valley Sewer Contract.* (14) Jan. 30.
Bodenfiltration und biologische Reinigung in Worcester, Massachusetts.* Erwin Neumann. (7) Serial beginning Oct. 17.
Ueber Luftfilter.* J. Kister und C. Finsterwalder. (7) Oct. 31.
Untersuchung über die Vorgänge in einer Warmwasserheizung während des Anheizens.* E. Lewin. (7) Oct. 31.
Versuchsergebnisse mit dem neuen Dampf- und Formalin-Desinfektionsapparat.* Otto Mayer. (7) Oct. 31.
Vergleiche über Emscher-, Kremer- und Stiagbrunnen, Neustädter Becken, biologische und chemische Klärung nach den Ergebnissen der Stuttgarter Versuchskläranlagen.* Emil Maier. (7) Serial beginning Nov. 7.

Structural.

Proposed Revised Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block. (110) Oct.
Reinforced Concrete Chimney at the Burnside Shops, I. C. R. R.* A. M. Wolf. (87) Oct.
The Work and Some Accomplishments of the Forest Products Laboratory, Madison, Wis.* Howard F. Weiss. (4) Dec.
Permanent Magnetism of Certain Chrome and Tungsten Steels. Margaret B. Moir. (Abstract from *Philosophical Magazine.*) (73) Dec. 25.
Shear and Problems Arising Therefrom. H. Kempton Dyson. (Paper read before the Concrete Inst.) (96) Dec. 31.
Architectural Acoustics.* Wallace C. Sabine. (3) Jan.
Floor Surfaces in Fireproof Buildings. Sanford E. Thompson. (Abstract of paper.) (55) Jan.
Reinforced Concrete Factory Buildings. F. W. Dean. (Abstract of paper.) (55) Jan.
Shear in Beams; Presentation of a General Formula and its Reduction to Special Formulas for the Common Simple Sections.* Ernst Jonson. (14) Jan. 2.
The Model Dairy Barn.* (From the *National Builder.*) (76) Jan. 5.
Knockdown Camp Buildings.* Charles E. Bee. (13) Jan. 7.

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Structural—(Continued).

The Philadelphia Ruling on the Design of Flat Slabs of Reinforced Concrete. Edwin Clark. (13) Jan. 7.
Piledriver with Traversing Leads for Driving Trestle Bents.* (13) Jan. 7.
Fireproof Ice-House at Lake Hopatcong.* (14) Jan. 9.
Paper-Covered Frames for Cold Weather Concreting.* A. A. Lane. (14) Jan. 9.
Restoring Concrete Buildings at Edison Plant.* (14) Jan. 9; (13) Jan. 14.
Grouting as a Method of Engineering Construction. (86) Jan. 13.
Structural Steel Supports for Large Steam Turbines.* (13) Jan. 14.
Tearing Down a 250-Ft. Reinforced-Concrete Chimney.* (13) Jan. 14.
The Behaviour of Metals Under Stress.* Fred C. A. H. Lantsberry. (Paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 15.
Comparative Exposure Test of Vehicles for Paint. Cloyd M. Chapman. (13) Jan. 15.
Methods of Excavation for Buildings.* A. B. McDaniel. (14) Jan. 16.
Routine Tests for Determining Strength of Mortars.* Harold Perrine. (14) Jan. 16.
San Francisco's New Auditorium.* (111) Jan. 16.
Design and Construction of the 435-ft. Steel Framed Tower of Jewels at the Panama-Pacific International Exposition.* Frank S. M. Harris. (86) Jan. 20; (14) Jan. 23.
Tests for Bond and Electrolytic Corrosion of Painted Reinforcing Steel.* (13) Jan. 21.
Test Results with Concrete Waterproofing Materials. (13) Jan. 21.
Sheet Metal for Church.* W. A. Douglas. (101) Jan. 22.
Suspended Beam Forms, Their Advantages and Disadvantages.* Sydney Diamant. (14) Jan. 23.
Treated Timber for Factory Construction. F. J. Hoxie. (Abstract of paper read before the Am. Wood Preservers' Assoc.) (14) Jan. 23.
Effect of Duration of Mixing on the Strength of Concrete. H. H. Scofield. (Paper read before the Indiana Eng. Soc.) (86) Jan. 27.
Test of Long-Submerged Hemlock Timber.* T. R. Lawson. (13) Jan. 28.
Wind Pressure Formulas and Their Experimental Basis. R. Fleming. (13) Jan. 28.
Factors of Safety in Steel and Iron Parts.* W. Trinks. (62) Feb. 1.
Note sur le Transformations Calorifiques de l'Energie du Choc. N. Davidenkoff. (93) Aug.
Zur Berechnung der Schubspannungen in gebogenen Stäben. Siegmund Fuchs. (48) Aug. 15.
Die experimentelle Ermittlung des Verlaufes der Stosskraft und die Bestimmung der Deformationsarbeit beim Stauchversuch.* H. Seehase. (48) Aug. 22.
Untersuchungen über die Wirkung von Anfangsspannungen in Nieten und Schrauben.* H. Kayser. (48) Sept. 19.
Das neue Haus des Vereines deutscher Ingenieure in Berlin.* (48) Oct. 10.
Bindung zwischen Asphalt und Beton.* (80) Oct. 17.
Die technischen Einrichtungen der Grossmarkthalle in München.* Karl Hauser. (48) Oct. 17.
Neuere Versuche über elastische Hysterese.* R. Grammel. (48) Nov. 28.
Oct. 24.
Untersuchungen über die Wärmeleitfähigkeit feuerfester Baustoffe.* E. Heyn. (80) Oct. 29.
Betonarbeiten bei Frost.* (80) Nov. 14.
Der Wellblechstoss.* H. Nitzsche. (48) Nov. 14.
Neuere Versuche über elastische Hysterese.* R. Grammel. (48) Nov. 28.
Eine eigenartige Eisenbetonpfahlgründung.* H. Schürch. (78) Serial beginning Dec. 5.
Der Ermittlung der ungünstigen Laststellung auf graphischem Wege.* Eduard Kreitmeyer. (78) Dec. 5.
Der Getreidespeicher der Aktien-Gesellschaft Lehnkering & Cie. zu Duisburg.* E. Rossmann. (78) Dec. 5.
Die Tragkonstruktionen des Rathaus-Neubaues in Mülheim a. d. Ruhr.* Pappit. (51) Serial beginning Sup. No. 1, 1915.
Die Ausnutzung des Materiales in gelochten Körpern.* A. Leon und R. Zidlicky. (48) Jan. 2.
Der Neubau der Markthalle in Karlsbad.* Franz Drobny. (78) Jan. 4.
Stoffänger aus Eisenbeton.* Willy Obrist. (78) Jan. 4.

Topographical.

Lake Traverses on Railroad Preliminary Work. J. A. Macdonald. (14) Jan. 9.
The Underground Survey of Cincinnati; Methods and Costs of Locating and Mapping Subsurface Structures Preliminary to Installing New Sewerage System.* O. E. Carr. (14) Jan. 9.
Permanent Survey Stations.* John H. Eggers. (103) Jan. 23.

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Water Supply.

Coon Rapids Low Head Hydro-Electric Development on the Mississippi River Near Minneapolis, Minnesota.* J. W. Link, M. Am. Soc. C. E. (4) Dec.
48-Inch Cast-Iron Force Main for Atlantic City, New Jersey.* L. Van Gilder. (59) Dec.
The Growing Value of Ground Water Supplies. L. J. Le Conte. (59) Dec.
Laboratory Control of Water Supplies.* Edward Bartow. (59) Dec.
Legal Decision in a Water Supply Pollution Case. J. Walter Ackerman. (59) Dec.
New Intake Tower and Tunnel at the St. Louis Water-Works. Edward C. Davis. (Paper read before the Engrs.' Club of St. Louis.) (1) Dec.
Note on Ortho-Tolidin Test for Free Chlorine. W. F. Montfort. (59) Dec.
Notes on the Contamination of a Drinking and Railroad Water Supply by Sea Water and the Removal of the Salt Water from the Reservoir. John R. Downes. (59) Dec.
Some Relations Between the Water Supply and Typhoid Fever in Washington, D. C.* John Gaub. (59) Dec.
Hydro-Electric Power Plant in Chile.* R. E. Graham Clark. (12) Dec. 25.
Service Reservoir Construction.* E. R. Matthews. (11) Dec. 25.
The Norfolk Experimental Rapid Sand Filter Plant.* J. G. C. Christie. (36) Jan.
Percolation and Upward Pressure of Water.* W. A. Mitchell. (100) Jan.
The Whitton Pumping Station, Ipswich Corporation Waterworks. Charles W. S. Oldham. (104) Jan. 1; (66) Jan. 19.
Electrically Operated Contractor's Plant for Building Kensico Dam.* A. W. Carroll. (14) Jan. 2.
Cleveland West Side Water Supply Tunnel.* (13) Jan. 7.
Dam and Log Slide, High Falls, Que.* Jos. Percy McRae. (96) Jan. 7.
Pipe-Thawing Motor Truck of the Columbus Water-Works.* (13) Jan. 7.
Hydroelectric Development at Rumford, Maine.* (27) Jan. 9.
Los Angeles Aqueduct Mistakes.* F. C. Finkle. (111) Jan. 9.
Water Turbine Characteristics; Method of Obtaining Characteristic Curves for High-Power, High-Speed Wheels, and Explanation of Their Application.* George H. Bancroft. (14) Jan. 9.
Scraping a Fifteen Inch Trunk Main at Batley.* J. C. Barrowclough. (Paper read before the Institution of Water Engrs.) (66) Jan. 12.
Design and Construction of the New Supply Conduit (Nepang) of the Hartford, Conn., Water-Works. (86) Jan. 13.
The City Tunnel of the Catskill Aqueduct.* Walter E. Spear. (13) Serial beginning Jan. 14.
Excavation for Foundation of Elephant Butte Dam.* E. H. Baldwin. (13) Jan. 14.
The Inverted Weir.* E. W. Rettger. (13) Jan. 14.
New Waterworks at Madras.* (12) Serial beginning Jan. 15.
Water-Softening by the Lassen-Hjort and the Permutit Processes.* (11) Jan. 15.
Chicago's New Tunnel and Pumping Station, Part of Comprehensive Waterworks Plan.* Horace S. Baker. (14) Serial beginning Jan. 16.
Development of Water Power on Public Lands. (14) Jan. 16.
Special Venturi Meter for Irrigation Water Measurement.* (86) Jan. 20.
Canadian Water Power.* (96) Jan. 21.
Direct-Driven and Geared Centrifugal Pumps, Ross Pumping Station, Pittsburgh.* (13) Jan. 21.
Hydro-Electric Power Development at Cedar Rapids, Que.* (96) Jan. 21.
Leakage from Cedar Lake Reservoir, Seattle Water-Supply.* Charles Evan Fowler. (13) Jan. 21.
Placing Concrete in Reservoir Walls by Compressed Air, Montreal.* (13) Jan. 21.
Rapid Water Main Construction, Medicine Hat, Alberta.* (96) Jan. 21.
The Cleaning of Water Mains.* J. F. Springer. (46) Jan. 23; (60) Jan.
Decay of Wood Pipes. (14) Jan. 23.
Bacteriological Standard for Drinking Water Supplied by Common Carriers. (From Report of Comm. of the Treasury Department.) (86) Jan. 27.
Design and Structural Details of Concrete Lining for Irrigation Canals.* Samuel Fortier. (86) Serial beginning Jan. 27.
General Principles Governing the Relocation and Mapping of Unchartered Portions of Water Distribution Systems. Carleton E. Davis. (Paper read before the Am. Assoc. for the Advancement of Science.) (86) Jan. 27.
Irrigation by Pumping at Del Rio, Texas, Detailed Description of Plant with Special Reference to Design and Operation of First Large American Humphrey Gas Explosion Pump.* Alexander Potter. (86) Jan. 27.
Notes on Representative Iowa Water Supplies. John H. Dunlap. (From *University Extension Bulletin No. 8*, Iowa Univ.) (86) Jan. 27.
A Study of the Failures and Partial Failures of Dams in Pennsylvania. (86) Jan. 27.
The First Large American Humphrey Pump.* C. C. Trump. (13) Jan. 28.
Lining the St. Louis Water Tunnel with Concrete by Means of Compressed Air.* E. C. Davis. (13) Jan. 28.

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Water Supply—(Continued).

Puntledge River Power Development.* (96) Jan. 28.
Construction of Cumberland Waterworks.* Frank H. Eastman. (14) Jan. 30.
West Fork Waterworks Dam at Fort Worth; Combined Earth and Concrete Dam.* (14) Jan. 30.
Neuere selbsttätige Stauanlagen der Stauwerke A.-G. in Zürich.* Schleusner. (48) Aug. 8.
Wasserturm aus Eisenbeton.* (80) Oct. 10.
Die Turbinen der A.-S. Rjukanfos und deren Untersuchung.* Ernst Reichel. (48) Serial beginning Oct. 31.
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*Illustrated.

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SUGGESTED CHANGES AND EXTENSION OF THE UNITED STATES WEATHER BUREAU SERVICE IN CALIFORNIA

REPORT OF A COMMITTEE OF THE SOUTHERN CALIFORNIA ASSOCIATION OF MEMBERS.*

BY GEORGE S. BINCKLEY, M. AM. SOC. C. E.,
AND CHARLES H. LEE, ASSOC. M. AM. SOC. C. E.

The Committee, appointed by the President of the Association on April 22d, 1914, to outline a plan for strengthening and extending the mountain climatological work of the United States Weather Bureau in California, so as to meet more widely and fully the needs of the industries and communities of the State relative to water supply, respectfully submits the following report.

Climatology is defined as that subdivision of meteorology which treats of the physical condition of the atmosphere during an extended period of time. Knowledge as to the climate of a place is obtained by a study of its continuous weather records—atmospheric pressure, temperature, rainfall, snowfall, direction and velocity of the wind, relative humidity, and evaporation—for a long period of years. The general relation of climatic factors to run-off and stream flow, as cause and effect, is so obvious, even to the layman, that no discussion is necessary.

* This report was presented to the Southern California Association of Members, Am. Soc. C. E., at its meeting of October 14th, 1914, and received the endorsement of the Association. It will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the Report in *Transactions*.

Throughout much of California and the Southwest the important streams have their sources in the mountain ranges bordering the valleys, and receive an almost negligible contribution from the valleys. In many districts arid valleys lie adjacent to mountain ranges receiving an annual precipitation of from 20 to 30 in. In such regions the gathering of climatic data, as a basis for the study of stream flow and water supply, should be carried on in mountains within the productive portions of drainage areas. This fact has been generally recognized throughout the West by irrigators, engineers, and some local officers of the United States Weather Bureau, ever since the commencement of irrigation development on a large scale.

The intimate relation which mountain precipitation, through stream flow, bears to the development and settlement of California is well shown by consideration of the communities and industries depending on reliable water supply. Agriculture, the most important industry of the State, to reach its greatest success, must generally be accompanied by irrigation. This has been fully demonstrated in Southern California and the San Joaquin Valley, and its truth is now recognized in the central and northern part of the State. The agriculturist, therefore, is fully as much interested in mountain climatology as related to water supply as he is in the atmospheric conditions immediately surrounding his crops. Hydro-electric power is rapidly taking a leading place throughout the State in manufacturing, transportation, and agriculture, as well as in supplying the physical needs of the inhabitants for light and heat. The reliability of the supply is largely dependent on stream flow, so that here, again, the average citizen is interested in mountain precipitation. Municipalities throughout the State depend for their very existence on mountain stream flow, for, if they do not derive their water supply directly from mountain run-off, as in the new Los Angeles Aqueduct system and the proposed Hetch Hetchy development, their water is pumped from underground reservoirs which, in turn, depend on precipitation and stream flow for replenishment. Finally, there is the subject of flood control and prevention which affects more or less the whole State and requires, to the fullest degree, definite and extended knowledge of mountain climatic data for its correct solution. In connection with this there is also the navigation of inland streams to be considered in the Great Central Valley of the State. The many

interests which would benefit by the collection, co-ordination, and publication of climatic data from the mountains are thus apparent.

The practical usefulness of climatic data is in forming a basis for the forecast of future stream flow variation. It is on the latter

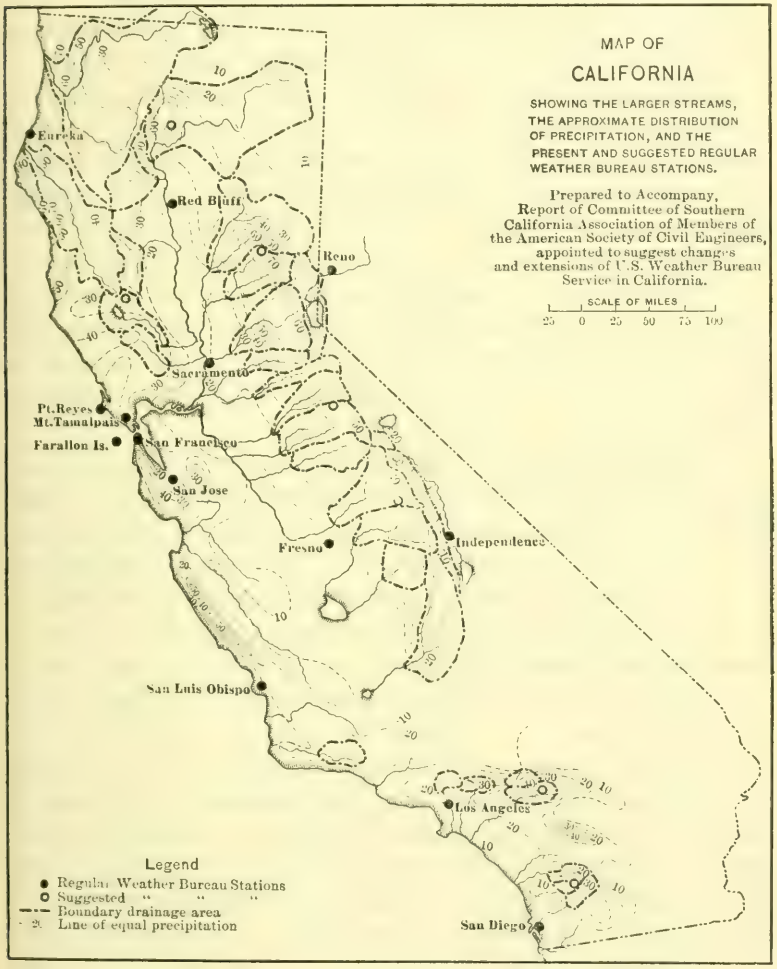
MAP OF
CALIFORNIA
SHOWING THE LARGER STREAMS, THE APPROXIMATE DISTRIBUTION OF PRECIPITATION, AND THE PRESENT AND SUGGESTED REGULAR WEATHER BUREAU STATIONS.
Prepared to Accompany, Report of Committee of Southern California Association of Members of the American Society of Civil Engineers, appointed to suggest changes and extensions of U.S. Weather Bureau Service in California.
SCALE OF MILES
25 0 25 50 75 100
Eureka
Red Bluff
Reno
Sacramento
Pt. Reyes
Mt. Tamalpais
Farallon Is.
San Francisco
San Jose
Fresno
Independence
San Luis Obispo
Los Angeles
San Diego
Legend
Regular Weather Bureau Stations
Suggested " " "
Boundary drainage area
Line of equal precipitation

FIG. 1.

information that the magnitude or practicability of irrigation projects, hydro-electric power systems, municipal water developments, flood-protection works, or any other large engineering work, depending on

water supply or stream flow, is based. Such data have also great value in the design of engineering structures for the control of water, such as dams, spillways, head-works, bridges, levees, etc. There is still another field that is beginning to be recognized, namely, in the business management of projects depending wholly or in part on snow storage. Where snow storage is the only source of supply, as is the case in many irrigation and small power projects, a predetermined knowledge of the season's supply would be of great value in making contracts for the delivery of water or power. Where the snow storage is supplemented by other sources, a previous knowledge of the quantity available makes possible a more complete economic planning for the development of an auxiliary supply. The work of the United States Weather Bureau in the Wasatch Mountains, Utah, has shown the practicability of making snow surveys preceding the period of melting which are sufficiently accurate to form the basis of contracts, between the irrigation companies and the irrigators, for the season's delivery of water in the valleys.* Such data from the Sierra Nevada streams of California would be of State-wide value.

It can be said, therefore, that comprehensive, accurate, and properly correlated data regarding mountain precipitation and related climatic data, such as temperature, wind movement, barometric pressure, humidity, and evaporation, are of the utmost importance in the permanent and economically efficient development of the State.

The extent to which such data are made available by the United States Weather Bureau in California will now be considered, first as to the geographic and vertical distribution and then as to the character of the observations.

The Bureau maintains stations of two classes: regular stations, fully equipped with standard recording instruments, in charge of trained observers; and co-operative stations, with incomplete instrumental equipment, observed without compensation, by public-spirited citizens. There are 13 regular stations in California, of which 6 are on the sea coast, 2 on the coastal plain, 4 in interior valleys, and 1 at Independence, Inyo County, at the western edge of the Great Basin, in an arid region. There are no stations in mountainous regions, or in fact in any region from which the local precipitation and run-off will ever be an important source of water supply. There are listed in

* *Engineering News*, May 29th, 1913, p. 1110.

the California Section of the Climatological Service of the Weather Bureau, Annual Summary for 1913, 192 co-operative stations, of which 17 (9%) are within 25 miles of the coast; 104 (54%) are in interior valleys and foot-hills; 16 (8%) are in the Great Basin and desert; and 55 (29%) are in regions of broken topography where the elevation exceeds 2 000 ft. Of the latter, 4 are above 6 000 ft. elevation, 7 are above 5 000 ft., and 21 are above 4 000 ft. The vertical limits of the most copious mountain precipitation in California are between 4 000 and 6 000 ft. elevation. Of the 21 stations above 4 000 ft. elevation, 13 are in the Sierra Nevada Range, the remainder being well scattered over the State. Of the 13 in the Sierra Nevada Mountains, 10 are concentrated in the group of counties: Butte, Nevada, Placer, Plumas, Sierra, and Alpine, which comprise only one-fifth of the length of the range. The other 80% of this highly productive drainage area has only 3 stations above 4 000 ft. elevation. The important area tributary to San Joaquin, Kings, Kaweah, Kern, and Owens Rivers has only one station, namely at Bishop Creek, Inyo County, at an elevation of 8 500 ft.

The character of the climatological observations made by the Bureau will be considered: first, as to numerical climatic data obtained; second, as to location and exposure of instruments; and third, as to qualifications of observers.

The following data are obtained at regular stations:

(1) Continuous air temperature records;
(2) Wet and dry bulb thermometer observations for relative humidity at 5 A. M. and 5 P. M.;
(3) Continuous precipitation records;
(4) Clearness of atmosphere;
(5) Continuous record of direction and velocity of wind;
(6) Continuous record of barometric pressure;
(7) Occasional water evaporation records;
(8) Miscellaneous observations.

Such data are all useful to the engineer as well as the climatologist, and if obtained under representative local conditions are of great value.

At co-operative stations served by volunteer observers the following data are obtained:

(1) Daily readings of maximum and minimum thermometers;

(2) Total precipitation each 24 hours as caught and observed in standard 8-in. gauges (snow catch usually computed as equivalent water in ratio 10 to 1), depth of snowfall in 24 hours measured on the ground, total depth of snow on ground at time of observation;

(3) Miscellaneous phenomena, such as occurrence of frost, coronas, thunderstorms, tornadoes, auroras, general phenomena of climate, and general character of the day.

The method of snow measurement prescribed at co-operative stations is rather unsatisfactory, both in catching and in reducing to equivalent water. Great improvement would result from the substitution of snow bins for the 8-in. rain gauges. It is a simple matter to cut and weigh cores from the snow on a bin floor. Such a procedure would eliminate the present elements of inaccuracy: first, as to the effect on snow catch of wind currents about the mouth of the 8-in. gauge; second, as to judgment of observer in ascertaining snow depth; and third, in the use of the approximate ratios of fallen snow depth to equivalent water.

The location of instruments at regular stations in large towns and cities (9 out of 13 stations) is on the top of high office buildings. This has a tendency to detract from the practical value of temperature, precipitation, and wind observations. As regards temperature, it is a frequently observed fact in Southern California that on winter mornings, when low growing crops are suffering from freezing temperatures, the official record will show much milder temperature. This can be explained, first, from the fact that the record is kept in an upper and warmer air stratum, and second, by convection and radiation from the steam-heated building on which the thermometer shelter is placed. Then, as regards precipitation and wind, tall buildings cause vertical and cross-currents and eddies, and these obviously affect the precipitation catch as well as the anemometer record. At the regular stations in small towns and at isolated points the location is usually on the open ground, and far better exposures are secured. At co-operative stations the thermometer shelter and rain gauge are usually on the ground, but the gauge is often very poorly placed with respect to obstructing objects such as trees, outbuildings, or houses.

The observers at regular stations are trained men who usually make a life study of meteorology, and are accurate and systematic in their

work. They are essentially indoor office men, however, and are not encouraged by the Washington office to get out into the field, observe conditions on the ground, and keep in close touch with local conditions and the work of co-operative observers. There is no such body of active, efficient, field observers, for instance, as is maintained by the Water Resources Branch of the United States Geological Survey. As a result of this policy, the Weather Bureau has in many respects failed to meet the plain needs of western communities and provide data of uniform reliability.

The co-operative observers are private citizens, without training in systematic and accurate observation, who volunteer their services, and receive no compensation. They place and observe rain gauges without official supervision or instruction. Instruments are often not understood, or are out of repair, readings are occasionally omitted, and the general character of the observations is not by any means up to a first-class standard. The obvious remedy for these defects is frequent personal supervision by trained men from the district central office. Nevertheless, the co-operative observers have filled a great need, and the Weather Bureau is to be commended for its activity in finding public-spirited citizens who are willing to undertake such work, and for the rapidity with which these records are made available to the public, and the convenient manner in which they are presented.

Summarizing the work of the Weather Bureau in California, as regards the gathering of mountain climatic data, it appears:

First.—That no regular stations are maintained within the productive drainage area of any important stream of the State, nor are any systematic scientific studies made of the subject;

Second.—That of the 55 co-operative stations maintained in mountainous regions, 21 are above 4000 ft. elevation, of which only 13 are in the Sierra Nevada Range, and 10 of the 13 are concentrated within about 20% of the area of the west slope;

Third.—That the co-operative stations are not equipped with instruments or men capable of gathering intelligently the data necessary for a comprehensive scientific study of mountain climatology as related to stream flow;

Fourth.—That the Weather Bureau does not maintain a corps of trained field observers capable of supplementing and keeping in touch with the co-operative observers;

Fifth.—That the placing of temperature, precipitation, and wind recording instruments on high buildings is practically without value to the agriculturist or engineer who attempts to use the data thus obtained.

In great contrast with the lack of regular stations in the mountains is their superabundance in the populated districts and along the coast. For example, within 50 miles of San Francisco there are 5 regular stations, San Francisco, San José, Point Reyes, Tamalpais, and Farallon. In the Great Valley there are 3 regular stations, Red Bluff, Sacramento, and Fresno; and along the south coast there are also 3 regular stations, San Luis Obispo, Los Angeles, and San Diego. At Independence, in a desert region possessing the characteristics of the Great Basin, there is maintained a fully equipped station which, because of its location, can have no practical value in solving the water supply problems of the State, unless supported by a station in the High Sierra to the west. Hence, there is evident a great inequality and inconsistency in the present distribution of regular Weather Bureau stations.

The Committee, after careful consideration of the facts as just stated and a study of the needs of the situation, presents the following outline as a working basis for extending the usefulness of the Weather Bureau:

I.—Establish at least 6 regular stations in the most productive mountain drainage areas, fully equipped with self-recording instruments and in charge of trained observers fitted for active outdoor field work. These stations should be, not only points at which climatologic data are collected, but also the headquarters for field observers having in charge local co-operative observers, and carrying on widely extended climatic surveys and special studies. If lack of funds prohibits the establishment of new stations, there is no reason why most of the existing stations could not be moved. Such action would be rather hard on certain of the Weather Bureau men, but would be of great benefit to the people of the State as a whole, and tend to distribute the benefits of the service more equitably. The locations of such stations suggested by the Committee are as follows:

1.—In the drainage area of McCloud or Lower Pitt Rivers;

2.—In the drainage area of the Feather or Yuba Rivers;

3.—In the drainage area of Stanislaus or Tuolumne Rivers;
4.—In the drainage area of San Joaquin or Kings Rivers;
5.—In the drainage area of Cache Creek above Clear Lake;
6.—In the drainage area of the Santa Ana River;
7.—In the drainage area of San Diego or San Luis Rey Rivers.

The conditions which should govern the location of these stations are: accessibility by wagon road or railroad from populated districts; accessibility in winter by skis or other means of travel for purposes of reading snow gauges and making snow surveys at points several days' journey distant; topographic features such as to insure the representative character of the station records; they should be in the zone of maximum rather than minimum precipitation for the drainage area; and, generally, should be where the facilities for the study of climatology as related to local water supply would be the best.

II.—Increase the number of co-operative stations in mountain areas now sparsely covered or not covered at all. In uninhabited areas, where neither mining settlements nor forest ranger stations are available, establish rain and snow gauges capable of holding a catch for an extended period.

III.—Organize a corps of trained field observers whose duties it will be to supervise personally the work of co-operative observers, and make special field studies, such as listed under IV.

IV.—Make special studies as follows, in as wide a territory as possible:

1.—Annual snow surveys in the drainage areas of Sierra Nevada streams;
2.—Snow movement subsequent to precipitation;
3.—The relation of temperature, forest trees, drifting, etc., to snow melting;
4.—The detailed relation between topography and precipitation;
5.—Water evaporation from lakes or reservoirs;
6.—Evaporation from snow;
7.—Establish, maintain, and observe snow and rain gauges at isolated points which can be reached only at long intervals;
8.—Any other investigations of practical value in the solution of the water-supply and flood-protection problems of the State.

The Committee has been frank in its criticism of the apparent fact that the principal meteorological observatories have generally been located rather more with reference to the comforts of a city than to secure the greatest efficiency and collect the most valuable data. This criticism, however, is not unfriendly. The staff of the Weather Bureau is uniformly courteous and obliging in furnishing whatever data are available; but, that these data have not a higher standard of reliability and greater practical value is due, the Committee believes, more to a lack of recognition of the great service that could and should be performed by the Weather Bureau, than to any reluctance on the part of those at its head to extend the scope and improve the value of its work.

In conclusion, the Committee does not hesitate to express the opinion that the questions involved in an improvement in the character of service rendered by the Weather Bureau may be discussed by engineers with the most complete freedom, and suggestions may be made without the slightest diffidence. Without in any way reflecting on the unquestioned scientific attainments of the officials of the Bureau, it is safe to say that the engineer who has to deal with the practical questions of water utilization is better qualified to indicate the source and character of meteorological data which will have the greatest real value, than the detached and abstract observer of meteorological phenomena.

The conclusions arrived at by the Committee are the result of mature consideration, and are now laid respectfully before the Association.

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CINDER CONCRETE FLOOR CONSTRUCTION BETWEEN STEEL BEAMS.

By Harold Perrine and George E. Strehan, Juniors, Am. Soc. C. E.

To be Presented March 3d, 1915.

Synopsis.

This paper treats of the special form of cinder concrete fire-proof floor construction between steel beams which is used extensively in New York City and elsewhere. It comprises a brief general discussion of the characteristics of the material, with reference to fire resistance, corrosion, strength, etc. The major portion, however, deals with the results of an extensive series of tests on cinder concrete, plain and reinforced, both in the form of cylinders and typical slabs, as a means of obtaining sufficient accurate basic data for use in compiling possible future regulations. The tests were conducted by the Department of Civil Engineering, Columbia University, partly in co-operation with the Bureau of Buildings, Borough of Manhattan, New York City.

Cinder Concrete in New York City.

History.—Prior to about 1885, the building laws of New York City allowed the use of practically nothing but burnt clay products as fire-resisting material. The construction of the arch filling or plat-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

form between steel or iron floor-beams was confined chiefly to heavy solid clay-brick segmental arches of specified dimensions. Gradually, as the general demand for "fire-proof buildings" increased, various digressions were made from the recognized standard floor to include many other forms of porous, semi-porous, or dense terra cotta tile and block, concrete, gypsum, and other mixtures, plain and reinforced in various ways, as well as several odd combinations of all these materials. The law contained a clause which permitted the use of "some equally good fire-proof material" as a substitute for brick arches, but the extremely vague manner in which this phrase was worded, the eager competition existing among manufacturers, and the fact that any new floor system, to be approved, must have been passed upon by the Board of Examiners, of which the membership was largely non-professional, brought about a rather chaotic condition in building affairs.

In order to eliminate the arbitrary method of awarding approvals, and to establish a standard test by which the quality of a fire-resisting floor construction might be judged, Superintendent Stevenson Constable, in 1896, had a number of test ovens erected, and invited leading manufacturers to supply typical specimens of their respective commercial products for the purpose of submitting them to a common test, or tests, which aimed to duplicate as nearly as possible, fire, water, and load conditions obtaining in actual conflagrations. The following is a report* of the test of the chambers and floor systems:

"The test kilns in which this series of tests were made were 11 x 14 feet, inside measurement, and 10 feet high from the upper grates to the floor system, which rested on the walls and formed the roof of the kiln. The walls were 12 inches thick, re enforced with buttresses and metal backstays. There were two sets of grates, the vertical distance between which was from 14 to 18 inches; air was admitted to same through openings in the walls at the ends of the ashpits. Chimneys 15 inches square were placed at each corner of the kilns. The floors tested were constructed as in actual building practice, the various arches were sprung between steel beams and the ceilings plastered as in a finished job. The wood flooring was not laid, but sleepers with concrete filling between them were in every case.

"The method of testing was to place a uniform load of 150 pounds per square foot on the central floor arch, then start a wood fire on the grates and keep it burning for a period of five hours, endeavoring

* *The Engineering Record*, September 18th, 1897.

to keep up a temperature of 2 000 degrees Fahr. for the last four hours. At the expiration of five hours water was applied to the interior of the structure for 15 minutes by the New York Fire Department, through a 1⅛-inch nozzle, under a pressure of 60 pounds per square inch; the first five minutes on the ceiling only, and the remaining 10 minutes on the walls and ceiling, principally the latter; then the top of the floor was flooded by a stream under lower pressure for five minutes. Subsequently the load of 150 pounds per square foot was removed from the central arch, and another of 600 pounds per square foot substituted; this was kept on for 48 hours, which concluded the test.

"The temperatures were taken just below the floor system by the pneumatic pyrometer made by Uehling, Steinbart & Co., of Newark, N. J., and also by placing various metals with known melting points in the kiln, in a position similar to that of pyrometer tube. Three iron rods upon which scales were fastened were placed along the center line of the central (loaded) arch in line parallel to the floor beams. One rod was placed at the center of the span of floor beams and one at each end; these scales on the rods were read by means of a transit giving the combined deflections of both floor beams and the arch between them."

Fourteen different types of fillings between steel beams were tested in this manner,* of which seven consisted essentially of plain or reinforced cinder concrete arches or slabs. In three of the remaining types the load-carrying members depended on this material for a portion of their strength.

From this series of tests it may be seen that the cheapness and lightness of the clinker resulting from the combustion of anthracite coal, used as an aggregate in this special form of concrete construction, would insure its increasing popularity, provided it met with service requirements. No disapprovals based on the results of these tests prohibiting the future use of any of the fourteen systems were issued by the Bureau of Buildings. No action was taken on two of the systems, one of which was of cinder concrete; and approvals were granted to twelve.

In 1902, after an interval of about 4 years, similar tests were conducted on various systems. This was practically coincident with the establishment of the Columbia University Fire Testing Station by Professor Ira H. Woolson. From that time forward the number of patented floors, and the consequent number of tests, increased

* *The Engineering Record*, September 18th, 25th, October 2d, 9th, 1897.

rapidly. In 1903 and 1904 the official tests averaged about one per month, but since that time there has been a gradual decrease in their number, and at present such tests are extremely rare. Exclusive of the 1896 series, 46 approvals have been issued by the Bureau of Buildings, based on fire, load, and water tests which have differed in prescribed severity to some extent from the original requirements. In 27 of these, or in more than one-half, cinder concrete has been the essential structural material of the filling between the steel beams.

It will be noted that an approval of a new floor system, through the agency of a report on a successful fire, load, and water test, permitted the use of the construction in question for spans not exceeding those included in the fire test, and to support safely a live load of 150 lb. per sq. ft. and no more. To obviate the necessity for additional fire tests on a system which had been approved as satisfactory in fire resistance, and for which an increase in span or live load, or both, was desired by the manufacturer, a load test was devised. This consisted in placing on a typical arch, similar to that for which approval was desired, and constructed for the purpose, a uniformly distributed load consisting of bags of sand, bricks, pig iron, paving blocks, or any other available material. This material was added until the treatment caused the total destruction of the arch, or until the desired load was attained. Approval was then granted for a safe live load per square foot equal to one-tenth of the maximum load per square foot placed on the arch. This method was used first in 1896 on an isolated section of one of the floor fillings after it had been subjected to a fire and water test. Bricks were used for the load, in this case, and were stacked solid. In fact, photographs of the earlier test loads are evidence of the extensive use of this material, as well as bags of sand or cement, and paving blocks stacked in a mass, interlocked when possible, with little if any indication of any attempt to prevent the consequent arching of the superimposed load. (Fig. 1.) Later, cast-iron pigs, steel billets, or similar material, cribbed and generally interlocked, were stacked in many cases to enormous heights, and slabs were approved for working loads, the accuracy of which is extremely doubtful. (Fig. 2.) These features are discussed quite fully in a recent paper* by Guy B. Waite, M. Am. Soc. C. E. A definite attempt to obtain greater accuracy was shown in the later

* "Cinder Concrete Floors," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1773.

FIG. 1.—LOADING WITH BAGS OF SAND. NOTE THE LONGITUDINAL SCANTLING UNDER TIMBERS, AND THE UNLOADED OR ANCHOR-SPANS OF CONCRETE. SOLID THROUGHOUT THE ENTIRE DEPTH OF I-BEAMS.

FIG. 2.—LOADING WITH PIG IRON STACKED SOLID. NOTE EVIDENCES OF ARCH ACTION AND CONSEQUENT TRANSFER OF LOAD TO SUPPORTING I-BEAMS.

FIG. 3.—LOADING WITH SEGREGATED PILES OF PIG IRON. NOTE THAT THE STACKS HAVE COME INTO CONTACT, DUE TO DEFLECTION OF SLAB.

FIG. 4.—LOADING WITH SEGREGATED PILES OF PIG IRON. NOTE THE DANGEROUS CONDITION OF THE STACKS.

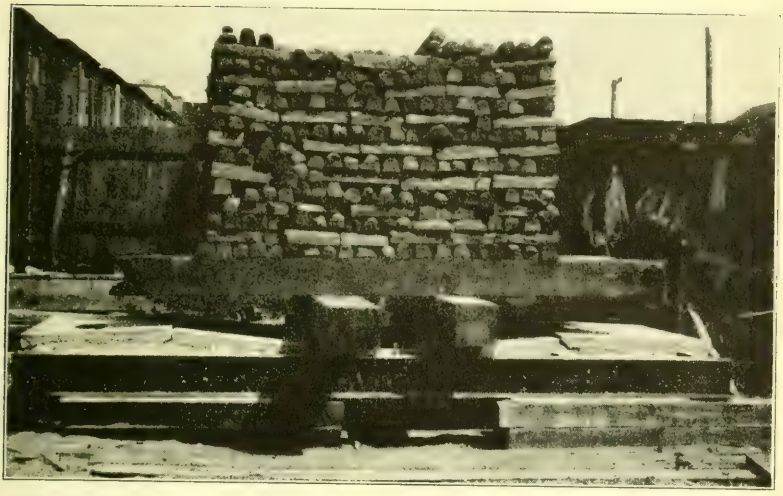


FIG. 5.—LOADING AT THE ONE-THIRD POINTS OF SPAN. ACCURATE AND COMPARABLE RESULTS ARE POSSIBLE WITH THIS METHOD.

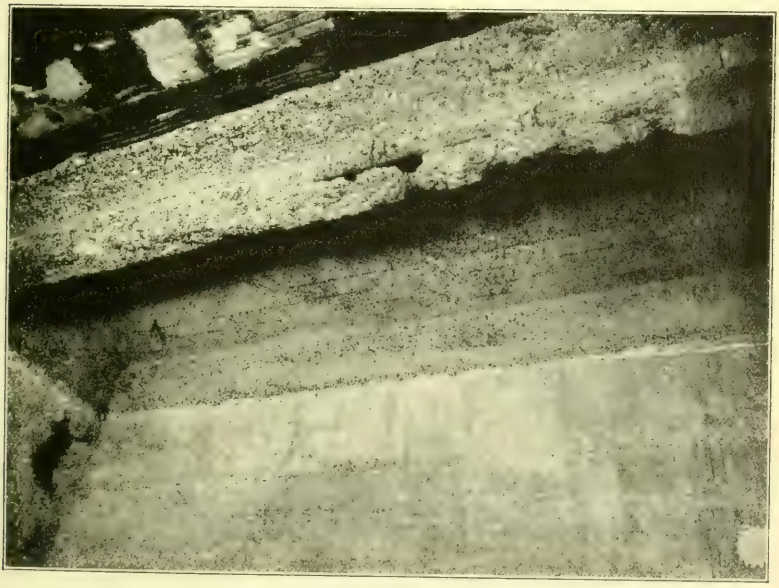


FIG. 6.—CONDITION OF 5 FT. 3-IN. FLOOR-SLAB OF 1:2:5 CINDER CONCRETE AT COMPLETION OF A STANDARD 4-HOUR FIRE TEST. SUPPORTING THIS IS SHOWN THE 12-IN. SIDE-WALL OF THE TEST STRUCTURE, ALSO OF CINDER CONCRETE.

tests, in which the pig iron was stacked in segregated piles; but, even when the piles are carefully placed, it becomes more and more difficult, as the loading progresses and the deflection of the arch increases, to continue without causing the piles gradually to come together at the tops. (Figs. 3 and 4.) It was only recently that this method was superseded in the Building Bureaus by one in which the load is applied by two equal concentrations symmetrically placed at the one-third points of the span. (Fig. 5.) This innovation simplifies the work of conducting a conclusive test, and renders possible the use of a testing machine, a hydraulic jack, or other mechanical means. It also obviates the necessity of applying actual weights, with the attendant danger to the laborers and the expenditure of time. To cause the failure of many of the test specimens, it often became necessary to handle excessive quantities of material.

As the earlier tests were along the line of investigative work in a new field, and were designed to meet immediate needs, one might expect the procedure and methods to be rather crude and faulty. The tests, however, served as a working basis for the earlier application of cinder concrete.

Of the 193 existing approvals in the Manhattan Building Bureau which are based on the fire, load, and water tests in conjunction with the special loading tests, 133 were issued for constructions consisting of reinforced cinder concrete.

Fire Resistance.

To any one at all familiar with the effect on cinder concrete of flames at high temperatures followed by the application of cold water at high pressure, little other evidence is necessary to prove its efficiency under this treatment. Experience is not confined merely to the results noted at the completion of standard tests, but is also based on examinations of buildings after having passed through actual conflagrations. The typical appearance of this material (mixture 1:2:5) after suffering the maximum punishment from fire and water may be seen clearly in Fig. 6, which is reproduced from a photograph taken from a point directly under and looking upward at the under surface of a floor slab which had passed through the standard tests, conducted by one of the writers. The portion of the wall, it may also be noted, forms one of the 12-in. side- or enclosure-walls

of the test chamber. The fire test, at 1 700° Fahr., lasted 4 hours, and was followed by the application of water for about 5 min., which produced the slight pitting noted on the slab surface. The wall, Fig. 7, on the other hand, had been subjected to seven such tests, aggregating 28 hours of fire.

Pitting on the surface of cinder concrete is due chiefly to the combustion of small particles of exposed unburned coal, in addition to the effect of de-hydration produced on all concrete. One of the chief arguments, derogatory to cinder concrete as a fire-resisting material, is that unburned coal, occurring in the aggregate in appreciable quantities, when burned out, will produce flaws and honeycombing, and consequent loss in strength. Portions of the slab shown in Fig. 7 were removed and inspected carefully. It contained a rather large percentage of unburned coal. At the very surface of the slab, where the flames had played directly on it, the coal had carbonized to a greater or less extent, but the body of the concrete remained practically intact, as shown by Fig. 8. In many instances pieces of hard, sharp, unburned anthracite were found within ½ in. of the exposed surface. The appearance of the fractured cross-section of this slab, on the other hand, seemed to indicate that actual de-hydration or other disintegration, as shown by the discoloration of the concrete, had occurred to the depth of from 1 to 2 in. of the original surface. Also, at one point there was a piece of charred wood, almost entirely consumed, which, judging by the size of the void in which it was found, had evidently been a part of a common lath. This void was about 2¼ in. from the surface which had been subjected to fire. This may be seen on Fig. 9, which also shows the location of a piece of brown paper which appeared to be slightly charred at its edges. A temperature high enough to anneal the cold-drawn wire reinforcement was certainly reached, as indicated by the reduction in its tensile strength and corresponding change in its elastic properties. Specimens of wire taken from the haunches and other unexposed locations showed a tensile strength of 74 100 lb. per sq. in., and this was reduced to 47 000 lb. per sq. in. for samples taken from the exposed portion of the slab proper. This reinforcement was placed 1 in. up from the bottom side of the slab. That the main body of the concrete possessed its original strength was evident from the fact that, as a result of actual tests on samples taken from the unexposed

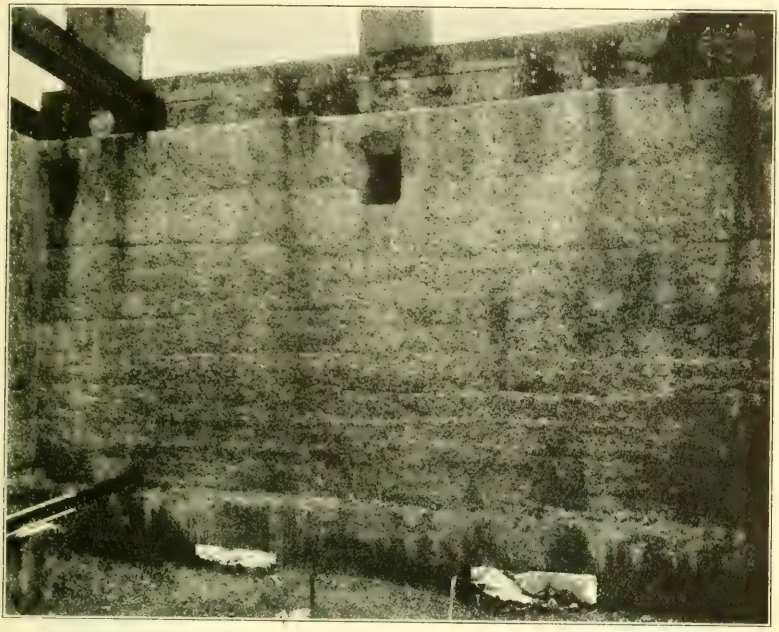


FIG. 7.—BACK-WALL OF STANDARD TEST HOUSE. THIS WALL WAS SUBJECTED TO 28 HOURS OF FIRE, AND NUMEROUS APPLICATIONS OF WATER UNDER PRESSURE.

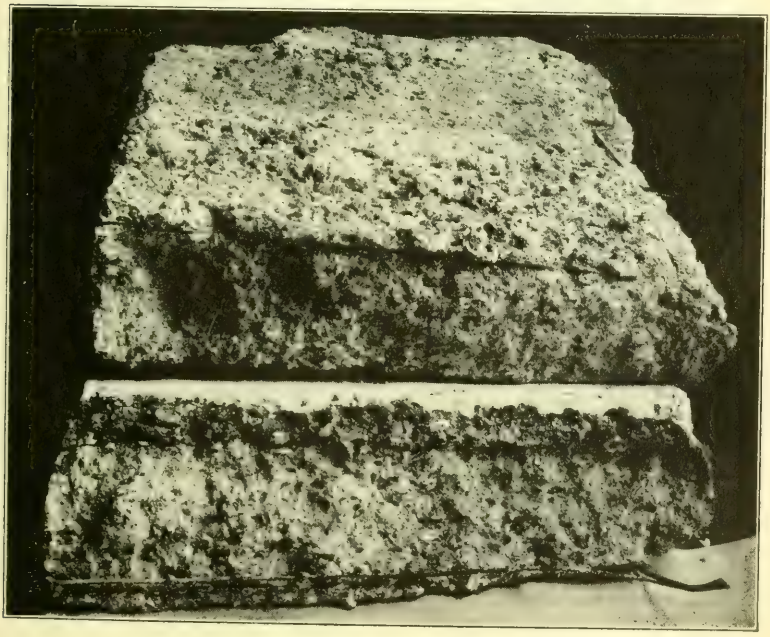


FIG. 8.—PORTIONS OF SLAB SHOWN IN FIG 6. UPPER PART SHOWS UNDER OR EXPOSED SIDE OF SLAB; LOWER PART SHOWS CROSS-SECTION.

FIG. 9.—PORTIONS OF SLAB SHOWN IN FIGS. 6 AND 8. VERTICAL SCALE INDICATES DISTANCE UP FROM ORIGINAL UNDER SIDE OF SLAB.

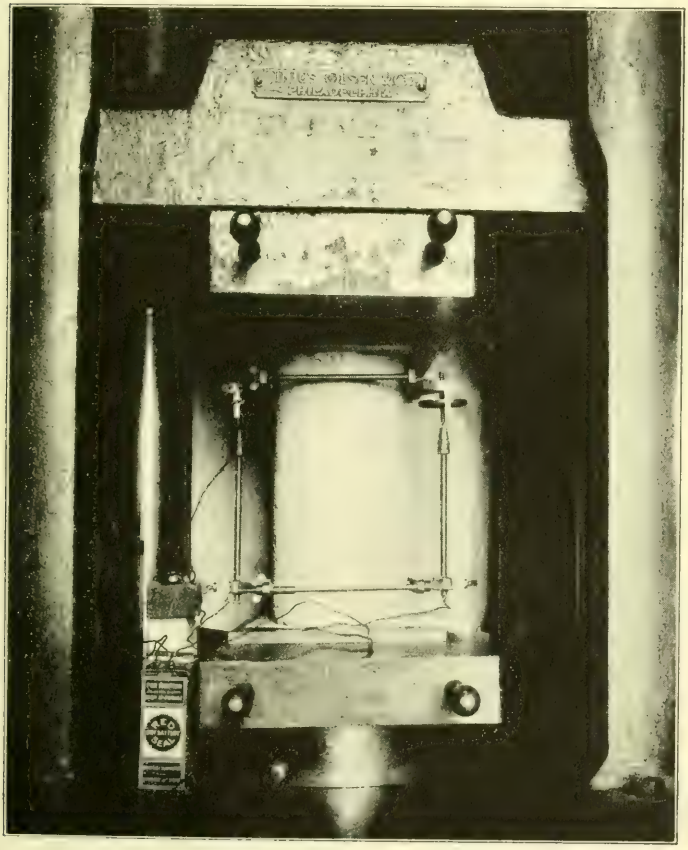


FIG. 10.—STANDARD CYLINDER IN TESTING MACHINE, SHOWING ELECTRIC CONTACT COMPRESSOMETER.

haunches and from the exposed slab, cubes of the former gave a compressive strength of 1 182 lb. per sq. in. normal to bedding, as against 1 273 lb. per sq. in. for cubes of the latter. Allowing for the shape of the test piece, and reducing the value obtained with cubes to that which might be expected with prisms having a vertical dimension equal to twice their width, it is found that the material subjected to fire had a compressive strength of 1 120 lb. per sq. in., and that material not thus treated showed a strength of 1 040 lb. per sq. in.

Corrosion.

Until recently, there has been much agitation relative to the question whether or not cinder concrete as such is directly responsible for the excessive corrosion of embedded steel. No particular reason was advanced as conclusive affirmative evidence of the truth of this idea, except the popular assumption that the sulphur in the aggregate became extremely active as a corrosive agent. For some time no other theory was advanced to explain the fact that steel or iron embedded in cinder concrete rusted to a greater or less extent, and it became rather a fad to condemn it for this reason, without intelligent inquiry into the causes. In a report submitted to the Structural Association of San Francisco, Mr. C. F. Wieland recommended that the use of cinder concrete be prohibited, due to "corrosive qualities and lack of strength". The doubtful wisdom of condemning, generally, a product which, within proper limitations, possessed many valuable characteristics, was criticized editorially in the same issue of the technical journal in which the report appeared.

A. L. A. Himmelwright,* M. Am. Soc. C. E., claimed, from experience gained in twenty-five inspections made during alterations of buildings, that more corrosion was noted where brick and stonework abutted against steel than in the case of any concrete. He also observed that there is no difference in the protection offered by stone or cinder concrete. Segregated sulphur, according to Mr. Himmelwright, is not possible, because it is largely vaporized during combustion, thus rendering the uniformly distributed sulphur extremely small.

Professor Charles L. Norton draws the following conclusions, in regard to the corrosion of steel in cinder concrete, as a result of

* *Engineering News*, 1906.

his well-known and widely-quoted series of experiments for the Insurance Engineering Station:

"Sulphur might [cause corrosion], if present, were it not for the presence of the strongly alkaline cement; but with that present the corrosion of steel by the sulphur of cinders in a sound Portland concrete is the veriest myth, and as a matter of fact the ordinary cinders, classed as steam cinders, contain only a very small amount of sulphur. There can be no question that cinder-concrete has rusted great quantities of steel, but not because of its sulphur, but because it was mixed too dry, through the action of the cinders in absorbing moisture, and that it contained, therefore, voids; and secondly, because in addition the cinders often contain oxide of iron which, when not coated over with the cement by thorough wet mixing, causes rusting of any steel which it touches.

"There is one cure and only one, mix wet and mix well."

The tests conducted by Mr. William H. Fox* at the Thayer School of Engineering, which were designed in a measure to throw light on the matter of corrosion, should not, in the writers' opinion, be considered at all conclusive. Primarily, the method of mixing the concrete was quite dissimilar to that common in building practice. The procedure consisted of spreading the cinders on a flat surface, with all the valuable void-filling fine material removed, and adding water to them until "no more was taken up". The dry sand and cement were then thrown on the wet cinders and the whole mixed together. It may readily be imagined that this manner of preparation of ingredients was not conducive to the making of dense or even typical concrete. The results, therefore, should be examined with this detail in mind. The variables introduced were, mix, consistency, method of tamping, and treatment during aging. Some specimens were cast dry and tamped, others wet and untamped, and still others wet and tamped. Several mixes, varying in richness from 1:1:3 to 1:3:6, were cast. Alternate water and air, or steam and air, curing were the treatments used. Steel of various shapes, embedded to a depth of 2 in. in all directions, corroded to a greater or less extent, regardless of the consistency, mixing, or method of casting the encasing concrete. Mr. Fox's conclusions are as follows:

"To secure a dense homogeneous cinder concrete, a thorough tamping is necessary. A rich mixture, either 1:1:3 or one in which the proportion of cement to aggregate is larger, should be used in all

* *Engineering News*, May 23d, 1907, p. 569.

cases. The greatest of care should be taken in mixing the materials, and it may be necessary to resort to the seemingly impractical method of coating the reinforcement with grout before placing in the concrete."

Mr. Fox also admits the severity of the tests.

As an exception to general experience, an instance of the effectiveness of porous cinder concrete was noted during the demolition of the Pabst Hotel,* New York City. It was there noted that corrosion of the steelwork existed only to a superficial extent. This fact becomes a matter of special interest when the type of floor system there used is considered in detail. The practice of the manufacturer of the arches in question was to deposit the concrete by throwing it from shovels on the wire mesh centering, omitting all tamping and stirring, the object being to obtain a porous product. This, it was claimed, would make it safe to deposit it in freezing weather, due to the absence of superfluous water.

Not so long ago, an instance of the inefficiency of porous cinder concrete used in a roof slab was brought before the public by the necessity for the removal of the roof of the train-shed of the La Salle Street Station, Chicago, Ill. In this case the cinder concrete, protected by a thin shell of stone concrete, reinforced with expanded metal, and exposed to gases and moisture coming from below, had to be renewed because of excessive corrosion.

Additional information relating to the corrosive tendencies of cinder concrete may be found in a series of articles and references listed in "A Bibliography of Corrosion of Iron and Steel in Cinder Concrete."†

An examination was made of the condition of the cold-drawn galvanized wire reinforcement, contained in the slab previously described, as to fire-resisting qualities. The floor system, after the completion of the fire, load, and water tests, had been directly exposed to the elements for more than a year without protection of any kind other than a 2-in. fill. In the region where the fire and water had pitted the surface, the wires, of course, were corroded. Where the concrete and reinforcement were still in practically their original condition, that is, in the haunches and anchorages, the wire was in much better condition. In many cases, long stretches of wire

* Report of Engineering Staff, Bureau of Buildings, New York City.
† *Engineering News*, April 18th, 1912.

were only affected slightly, despite the undue severity of the exposure. Corrosion was most apparent at the junction points where the lateral wires crossed the longitudinals, and seemed generally to have its beginning in the pocket or crevice thus formed in the concrete.

It is the writers' opinion that the corrosion of steel embedded in cinder concrete is still a matter of conjecture. The original reason for its existence was due, in all probability, to the idea prevailing, formerly, that concrete must be placed dry and religiously tamped and rammed in order to attain maximum density.

Present-day knowledge has led practice in many cases to the opposite extreme, with the result that concrete is often deposited too wet; but, cinder concrete, due to the very form of the aggregate, should not be deposited dry. By giving the well-mixed material a wet and viscous consistency, and constantly stirring it, the mortar is able to coat the reinforcement thoroughly and flow readily into the interstices of the larger clinkers, a condition which cannot obtain with dry material, no matter to what extent it is rammed. This practice, more than anything else, will tend toward the reduction of corrosion, which should not exist to any greater degree in cinder than in stone concrete, if proper care is exercised. This opinion is based on experience gained with the use of anthracite coal cinders. Western practice seems to indicate that, with the use of bituminous coal cinders as an aggregate, extreme and unexplained cases of corrosion have occurred.

Strength and Other Characteristics.

Cinder concrete as used in New York City is made with ingredients of widely differing characteristics. Materials used as the coarse aggregate vary by all degrees from the hard, vitreous clinker classed as "steam cinders" resulting from the combustion of anthracite coal, which is often free from ash and other foreign matter, to the almost useless, soft, flaky ash such as is commonly obtained from domestic coal stoves. Due primarily to the impossibility of adequate inspection, arising from the lack of a sufficient number of officials engaged in these duties, the resulting concrete is about as variable in quality as the available ingredients.

To make proper allowance for the variableness of the aggregate and for the workmanship, a factor of safety of 10 has been required

by the Building Bureau authorities in all load tests on cinder concrete floors.

The doubtful economy of introducing such a large factor of safety on selected test material, with resulting wide variation in the commercial product, should militate against a continuance of this method of control. The question then arises whether or not the product is adaptable to more scientific regulation, that is, can it be safely designed? An answer to this question can be found only in experience and knowledge gained by systematic research.

Present Investigation.

This absence of accurate knowledge of cinder concrete has led to the inception of an investigation, together with a series of experiments by the Department of Civil Engineering of Columbia University, partly in co-operation with the Bureau of Buildings, Borough of Manhattan, New York City.

General Scheme.—The investigation, instituted during the spring of 1913 and now in progress, is based on the following general scheme:

1.—The determination of the compressive strength, with elastic and other properties, of typical New York cinder concrete, cast under working conditions, as obtained in actual practice;

2.—A study of the behavior of typical cinder concrete slabs under load, as affected by the character and percentage of reinforcement, thickness, span length, age, and local conditions, for the purpose of aiding in theoretical design or establishing a standard loading test;

3.—An attempt to recommend specifications tending toward an improvement in the product, that is, greater density, uniformity, etc.

Typical New York Material.

Tests for compressive strength and elastic properties were conducted on standard cylinders, 16 in. high and 8 in. in diameter, as recommended by the Joint Committee. Galvanized-iron forms, having these interior dimensions, were taken to buildings in course of construction in various parts of Manhattan, and 40 cylinders were cast at each location.

All specimens were made by men familiar with the casting of test cylinders, and under the personal supervision of a Building Bureau official. The mixture in the specimens was identical with

that going into the floors at the time of arrival, no advance notification having been sent to the contractor of the intention to take samples. The various methods of handling, tamping, etc., were carefully noted. Test pieces were taken from five buildings selected from three arbitrarily chosen zones, the intention being to find local variations, if such existed. Samples of the component materials were selected from the cement, sand, and cinders, in each case, and tests and analyses of these were made. The writers wish at this point to acknowledge the cordial co-operation of Mr. J. F. Davis, of Robert W. Hunt and Company, in furnishing chemical analyses of all materials used in this investigation.

The cylinders were allowed to remain for several days under the atmospheric conditions obtaining on the mixing floors of the buildings in which they were cast, after which time they were removed to the Columbia Laboratories and stored in air at ordinary room temperature, until tested. During the early aging they were sprinkled occasionally. It was thought that this method of storage duplicated as nearly as possible the conditions under which the floor slabs themselves are cured. In many cases, due to their thinness, the slabs dry out quite rapidly. All cylinders were faced at both ends with plaster of Paris, before testing.

From each of the sets of 40 cylinders 10 were tested respectively, at 1, 2, 6, and 12 months. Fig. 10 shows a standard cylinder in the testing machine. From Table 1, showing the results of these tests, data on material from Building No. 5 have been purposely omitted. The latter was one of the new Columbia dormitory buildings, then in process of construction, and the contractor was aware of the intention to select samples.

Zone *A* applies to that region of Manhattan occupied for the most part by apartment houses.

Zone *C* is the down-town office building region.

Zone *B* refers to that large section lying midway between Zones *A* and *C*, and in which the new construction consists mainly of loft and mercantile buildings.

The arbitrary choice of zones was made, not only for the purpose of obtaining material from different types of buildings in process of construction, but also to study the effect of the conditions occurring at the source from which the cinders were obtained; for, in many

cases, the existence of oil, injurious chemicals, certain by-products of manufacture, and other impurities are important factors detrimental to the resulting concrete.

TABLE 1.—DATA RELATING TO CINDER CONCRETE AS USED IN FIRE-PROOF FLOORS IN NEW YORK CITY.

Zone	A_1	B_1	B_2	C
Mix	1 : 2 : 5	1 : 1 : 5	1 : 2 : 5	1 : 2 : 5
Method	Continuous mixer. Coltrin.	By hand turned twice.	Batch mixer.	Mixer, Ransome.
Cement	Alsen.	Dragon.	Vulcanite.	Atlas.
Sand	Typical Long Island Bank Sand, North Shore.			
Cinders	Anthracite. Ice plant.	Local hotel steam plant.	Local.	Local office building steam plant.
Weight, in pounds per cubic foot	107	100	107	109
One-month test:				
Crushing strength, in pounds per square inch	407	507	818	980
Modulus of elasticity, in pounds per square inch	924 600	857 400	1 230 000	1 492 000
Two-month test:				
Crushing strength, in pounds per square inch	701	662	1 254	1 035
Modulus of elasticity, in pounds per square inch	1 134 000	1 030 000	1 740 000	1 428 250
Six-month test:				
Crushing strength, in pounds per square inch	933	754	1 744	1 478
Modulus of elasticity, in pounds per square inch	971 000	1 050 000	1 348 000	1 276 000
One-year test:				
Crushing strength, in pounds per square inch	913	813	1 465	1 475
Modulus of elasticity, in pounds per square inch	993 000	956 000	1 200 000	1 320 000

Each value in Table 1 is an average obtained from 10 samples, except in the case of B_2, in which the total number of specimens tested was 30 instead of 40. This was due to the fact that about three-quarters of the material used was mixed under actual working conditions, and the remaining 10 cylinders were cast from a mix prepared by the contractor's foreman, who was aware of the use to which the test specimens were to be put. His evident zeal in the matter resulted in extremely high compressive strengths, and therefore these were excluded from the averages.

The modulus of elasticity was determined at a point on the elastic curve corresponding to one-fourth the ultimate strength.

The writers believe that Table 1 contains sufficient data to prove conclusively the variableness of cinder concrete as used at present in Manhattan. The strength of Sample A, 407 lb. per sq. in. at an age of 1 month, may be taken as typical of concrete resulting from the use of low-grade cinders, despite the fact that it was machine-mixed.

B_1 was found to have a strength of 507 lb. per sq. in., after a similar interval of time. In this case, the use of good cinders and a richer mix than is required did not offset the poorer results almost always obtained by ordinary hand-mixing. Sample B_2 approaches, and C about equals, good practice in cinder concrete construction at the present time. Fig. 11 shows fractured cylinders cast from typical New York materials.

Samples B_1, B_2, and C represent cinders of approximately the same gradation in size as reported in Table 6, Appendix I. The complete mechanical analyses of both the typical concrete aggregate and the test slab aggregate are given in Tables 6 and 9. The mechanical analysis of Sample A shows a greater preponderance of fine material in this cinder than in the other three samples. The conclusion as to poorer results being obtained with hand-mixing appears to be justified in view of the detailed tests of the component materials reported in the Appendices.

Behavior of Typical Slabs Under Load.

To date, tests have been conducted on 58 slabs, together with tests on corresponding cylinders and component materials.

Component Materials.

Cement.—The cement used in the 1913 series was the Dragon Portland delivered to the contractor for the fire-proof floor construction of the new dormitory building, Furnald Hall, Columbia University.

In the 1914 series the cement used was Dragon Portland furnished through the courtesy of the Lawrence Cement Company.

The results of the detailed tests are given in Appendix II.

Sand.—The sand used in both instances was the typical bank material obtained from the north shore of Long Island. Its physical properties are given in Appendix II.

Cinders.—For the 1913 series, the cinders were a typical commercial product furnished, by a dealer in this material, to the contractor for the floors of Furnald Hall.

The cinders for the 1914 series were obtained from the Columbia University power plant.

The physical properties of both aggregates appear in Appendix II.

Reinforcement.—The steel reinforcement consisted of triangular

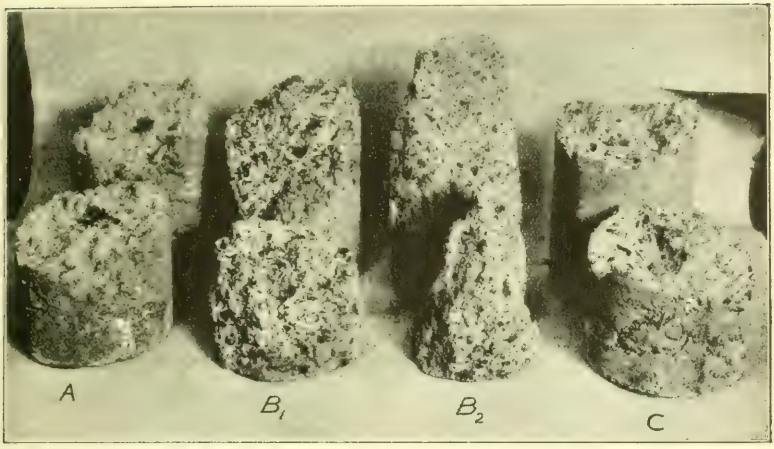


FIG. 11.—FRACTURED CYLINDERS CAST FROM TYPICAL NEW YORK MATERIALS.
A.—POOR MATERIAL CONTAINING LARGE PERCENTAGE OF DUST AND LARGE CLINKERS.
B_1.—FAIR MATERIAL; BUT CONCRETE IS POROUS.
B_2.—GOOD CONCRETE.
C.—GOOD CONCRETE.

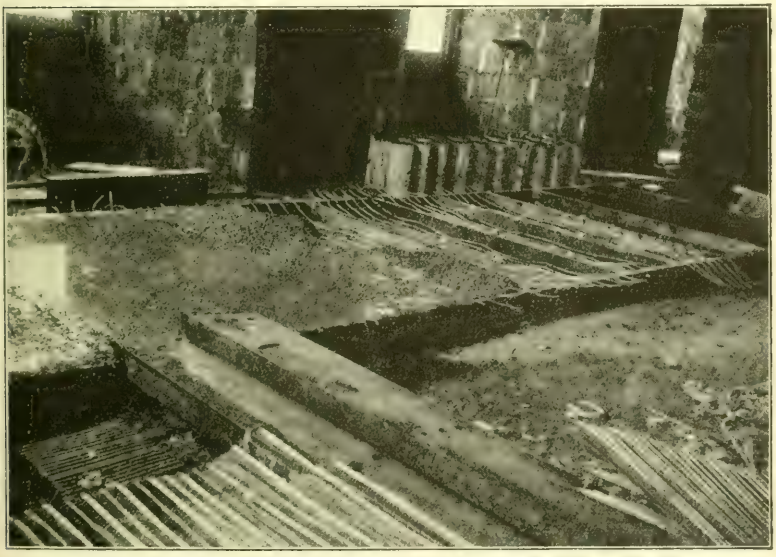


FIG. 12.—CASTING CYLINDERS AND SLABS, 1913 SERIES, DORMITORY BUILDING, COLUMBIA UNIVERSITY.

wire mesh generously furnished by the American Steel and Wire Company, expanded metal kindly submitted by the Expanded Metal Engineering Company, and the following types obtained in the open market: Electrically welded wire mesh, triangular wire mesh, expanded metal, plain round, plain square, and twisted square, rods.

TABLE 2.—SCOPE OF TESTS ON SLABS.

CONCRETE.			NUMBER OF SPECIMENS.						
			Slabs (All 22 in. wide).						
Mortar.	Aggregate.	Consistency.	Restrained.	Simple.	Span, in feet.	Thickness, in inches.	Cylinders, 8 by 16 in.	Reinforcement	Variables.
1913 SERIES									
Dragon cement.	Typical Long Island sand.	Commercial cinders.	Wet.	6	10	6	4	20	Triangular wire mesh. Welded wire mesh. Expanded metal.	Span; Thickness; Condition of end restraint; Percentage and character of reinforcement.
			Wet.	6	6	8	4	15		
			Medium.		3	8	5			
1914 SERIES									
Dragon cement.	Typical Long Island sand.	Commercial cinders from Columbia University steam power plant.	Viscous.	3	3	7	4	10	Triangular wire mesh. Expanded metal. Round rods. Square rods. Twisted square rods.	Span; Thickness; Condition of end restraint; Percentage and character of reinforcement.
			do.	3	3	6	4	10		
			do.		14	5	3 and 4	10		
			do.	(1—Special.)		7	4			
							Total cylinders	65		
								Restrained	18	
								Simple	39	
								Special	1	
								Total number of slabs	58	

Concrete.—The concrete in all cases consisted of 1 part Dragon cement, 2 parts Long Island sand, and 5 parts steam cinders, all by volume.

For the 1913 series, the materials were brought to a continuous mixer, mixed for about 1 min., deposited in wheel-barrows, transported about 200 ft. and dumped into the forms provided for the purpose,

and following throughout, a procedure exactly similar to that used in casting the actual floors of the building. Fig. 12 shows the slabs and cylinders in process of construction. The concrete was furnished through the courtesy of the Department of Buildings and Grounds, Columbia University. The consistency of the concrete was wet in some slabs and medium in others.

The materials for the 1914 series were mixed in the ¼-cu. yd. Blystone batch mixer of the Columbia Concrete Laboratory, which is used for most of the investigative and commercial work of this kind. The cement, sand, and cinders, carefully measured by volume, were deposited dry in the machine. Water from a hose connecting with a measuring tank was played on the ingredients as the mixing went on, the percentage of water being practically constant throughout the series. The process of mixing occupied about 2 min. per batch. The concrete was taken to the forms in wheel-barrows, and when deposited was of a wet, viscous consistency. It was stirred and pushed into the spaces between the wires or rods with narrow pieces of scantling, the material being too wet to permit of tamping, as the term is generally understood. An excellent, dense, homogeneous concrete was thus obtained. The casting of restrained and simple slabs is shown in Figs. 13 and 14.

The slabs for the most part remained in the room in which they were cast until the time of test, and were sprinkled occasionally during the early periods of aging. Cylinders cast at the same time received similar treatment.

Method of Construction.

The condition of end restraint and other details of construction of that type of arch styled in this investigation as "Restrained" may be best understood from an examination of Figs. 13, 15, and 19. In this construction wire mesh reinforcement was used exclusively, the slabs being in all cases 22 in. wide. The "free" slabs, or simple beams, all 22 in. wide, were cast as shown in Fig. 14. The uniform width of 22 in. was assumed in order to facilitate the testing of the simple beams in a testing machine, this being the maximum width possible, due to the spacing of the screws. Arches commonly known as "flat ceiling" or "bottom flange construction" have not yet been investigated.

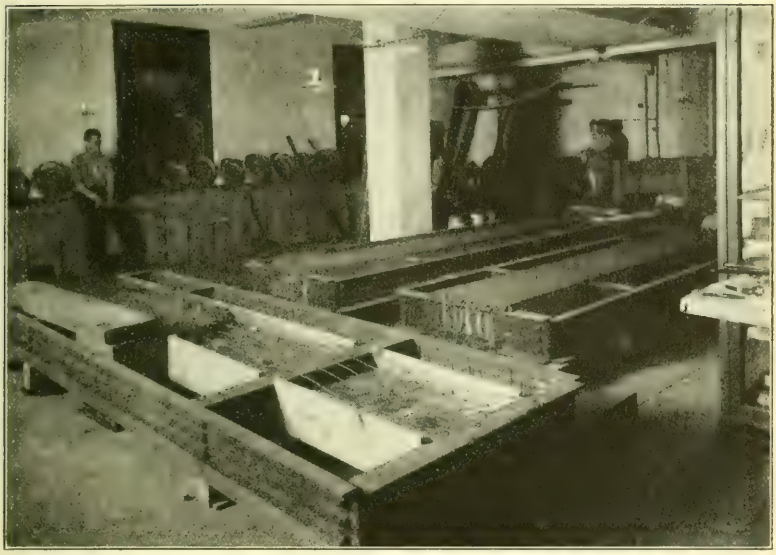


FIG. 13.—CASTING "RESTRAINED" SLABS, 1914 SERIES.

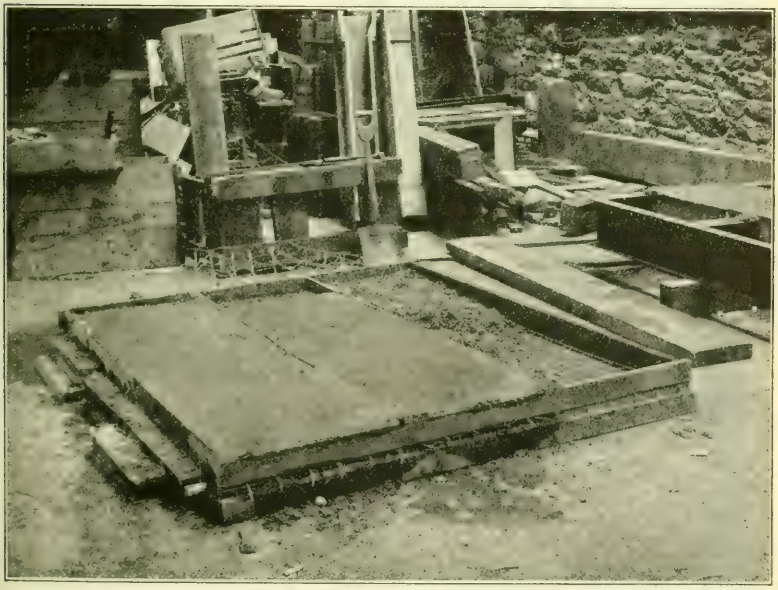


FIG. 14.—CASTING "SIMPLE" SLABS, 1914 SERIES.

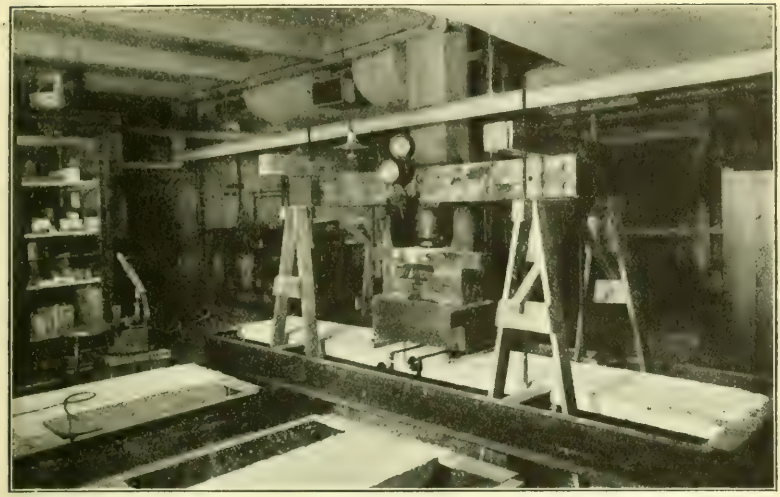


FIG. 15.—TESTING "RESTRAINED" SLAB WITH HYDRAULIC JACK. NOTE GAUGES AND RECORDING INSTRUMENTS IN PLACE.

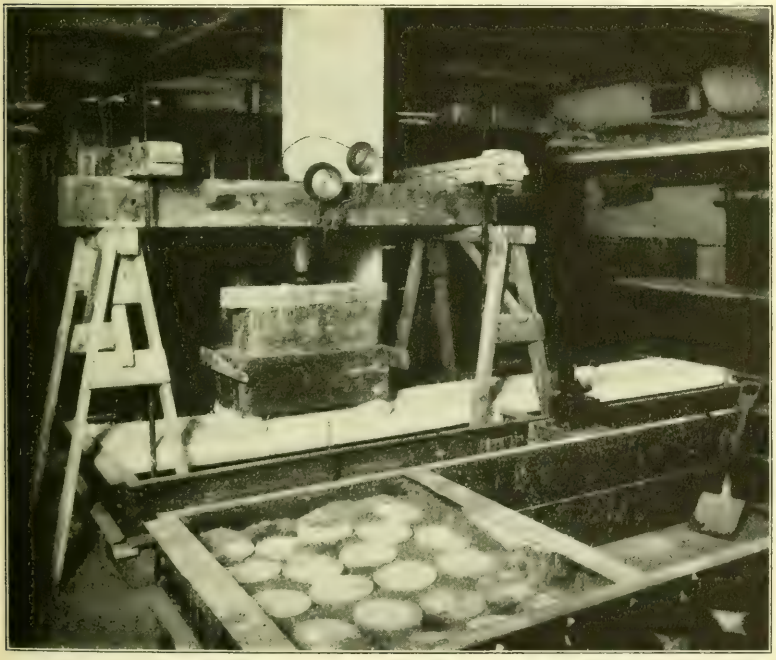


FIG. 16.—TESTING "SPECIAL TYPE," REPRESENTING END-BAY CONSTRUCTION.

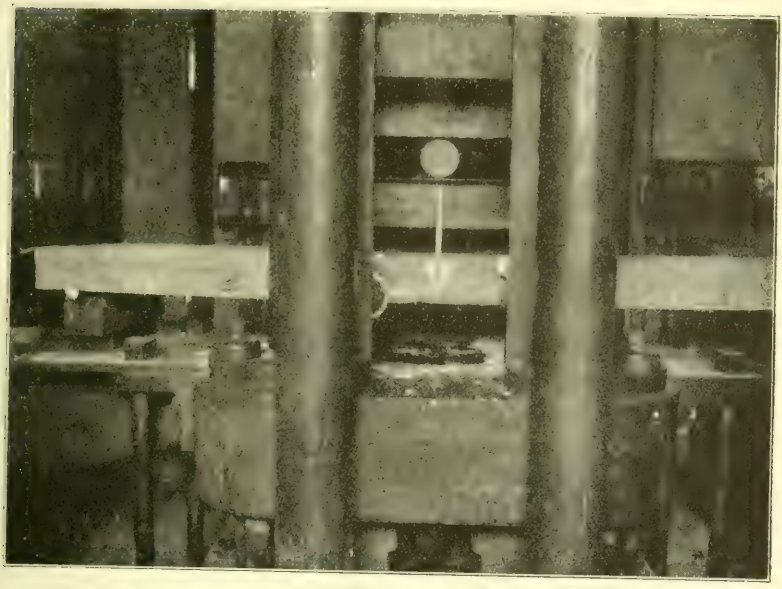


FIG. 17.—LOADING "SIMPLE" SLAB IN LABORATORY TESTING MACHINE.

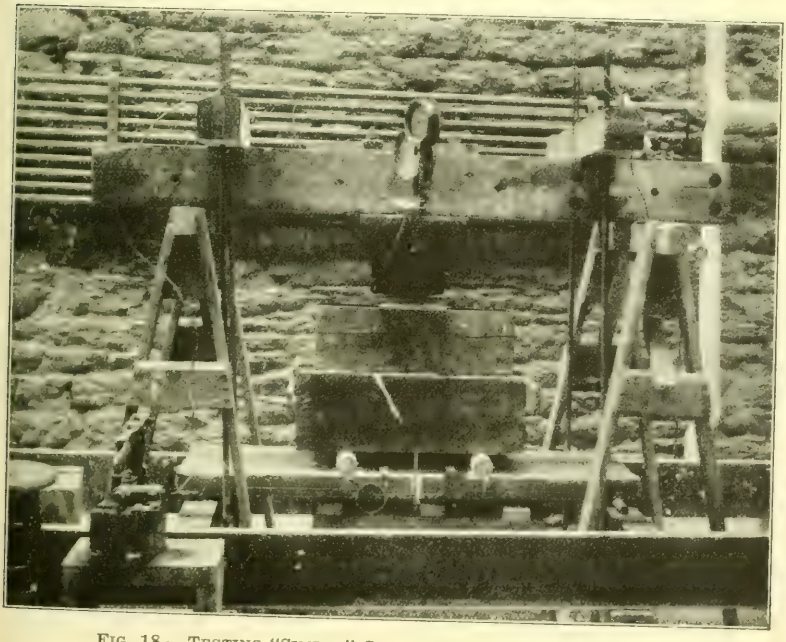


FIG. 18.—TESTING "SIMPLE" SLAB WITH HYDRAULIC JACK.

Methods of Testing.

Fig. 15 shows clearly the arrangement used in loading all the restrained arches. A hydraulic jack and hand-pump were used in applying the load, the latter being indicated on the gauges shown. The jack and attached gauges had been previously calibrated as a unit in a testing machine of known accuracy. The load was applied in increments of 5 lb. gauge pressure, corresponding to about 250 lb. of actual load.

The "Special" type mentioned in the table of results (Table 3) is shown in Figs. 16 and 19, the loaded span representing conditions obtaining in actual construction at a point where the slab abuts against an outside wall, or an enclosure of a vertical opening, and where proper end restraint is lacking.

Simple slabs were tested, in part, as shown in Fig. 17, resting on rollers placed on the platen of a testing machine or by the jack, as in Fig. 18.

Deflections were read at each side to the nearest hundredth of an inch by a steel scale, set in plaster of Paris at the mid-point of the span, intersected by a wire stretched over nails driven into the sides of the slab, at points directly over the roller supports in the case of simple slabs, or directly over the flanges of the supporting I-beams in the case of restrained slabs.

The strain produced on the tension side of the slabs was measured on 10 or 12-in. gauge lengths between two points symmetrically placed in the surface of the slabs, in the plane of the reinforcing steel which, in all cases, was cast 1 in. up from the bottom surface of the arch. In the case of restrained beams, the reinforcement was turned up at the one-fifth point of the span, in order to pass over the supporting I-beams.

Compressive strain was measured on a 10 or 12-in. gauge length on the sides of the slabs, $\frac{1}{2}$ in. down from the top surface of the arch, and afterward corrected for the corresponding probable extreme fiber strain.

Tensile and compressive stresses were computed from these measured strains, assuming the modulus of elasticity of steel to be 30 000 000 lb. per sq. in., the proper modulus for the concrete depending on compression tests of corresponding material.

Results.

Table 3 is a statement of the results of tests to date on slabs.

Computations and Discussion.

Form of Test Specimens and Applied Load.

Previous to the inception of the present series of tests, a thorough study was made of all records of load applications on cinder concrete

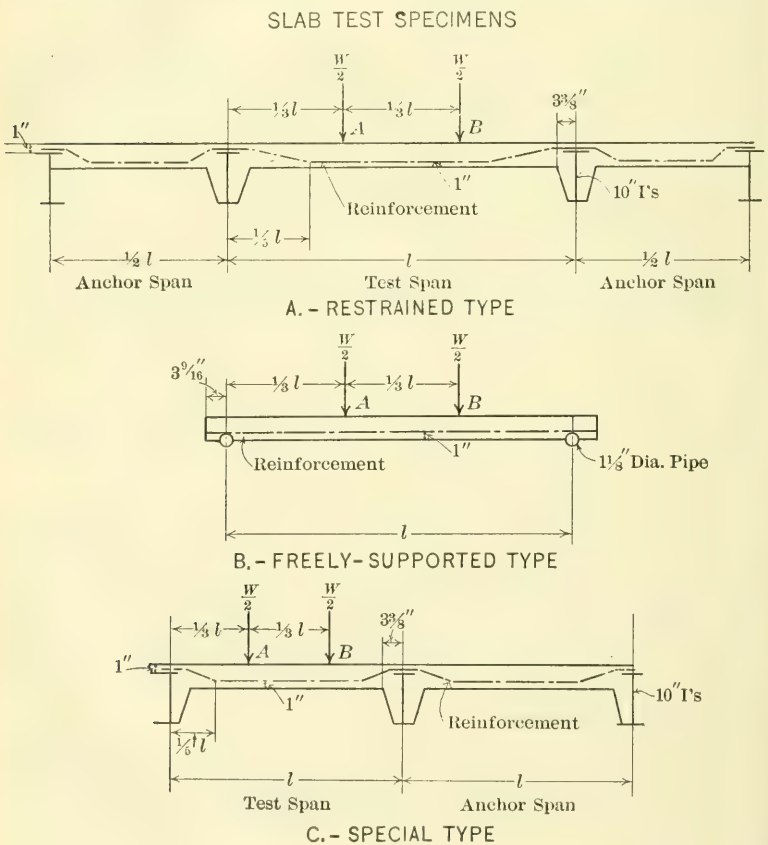


Fig. 19.

floor filling between steel girders; and all such tests that came under the direct observation of the writers were carefully inspected. In all the tests conducted during the years prior to 1911, the mode of installation, as regards anchorage of the reinforcement and provision

TABLE 3—RESULTS OF SLAB TESTS.

Slab No.	Reinforcement.	Span.	Thickness. Net.	Thickness. Gross.	End condition.	Percentage of steel.	Age, in weeks.	(jd)	(kd)	(n)	Maximum applied load, in pounds.	Maximum deflection, in inches.	Estimated weight, in pounds.	Equivalent load, in pounds per square foot.	Uniform dead load, in pounds per square foot	Equivalent total uniform load, in pounds.
C 7-ft.																
1	Triangular wire mesh. Area,	7′ 1″	3.16	4.16	R.	0.26	4	2.85	0.98	26	8 700	1.22	481	894	37.0	931
2	0.173 in.	7′ 1″	3.08	4.08	R.	0.27	4	2.77	0.95	26	8 175	1.70	473	839	36.4	875
3		7′ 1″	3.16	4.16	R.	0.26	4	2.85	0.98	26	7 475	1.86	481	768	37.0	805
7-ft.																
1		7′ 1″	3.13	4.13	F.	0.26	4	2.83	0.97	26	2 900	1.78	478	298	36.8	335
2	" " " " "	7′ 1″	3.53	4.33	F.	0.23	4	3.20	1.03	26	2 650	1.63	525	276	40.4	316
3		7′ 1″	3.33	4.33	F.	0.25	4	3.00	1.00	26	2 750	2.15	501	283	38.6	322
7-ft. Special.	" " " "	7′ 1″	3.00	4.00	F. & R.	0.27	4	2.70	0.93	26	4 675	0.63	463	481	35.6	517
C 6-ft. 1		6′ 0″	2.90	3.90	R.	0.28	4	2.61	0.84	22	8 675	1.25	386	1 051	35.1	1 086
2	" " " " "	6′ 0″	2.97	3.97	R.	0.28	4	2.67	0.86	22	10 075	1.55	393	1 220	35.7	1 256
3		6′ 0″	3.27	4.27	R.	0.25	4	2.95	0.95	22	10 920	0.89	423	1 325	38.5	1 364
6-ft. 1		6′ 0″	3.02	4.04	F.	0.27	4	2.72	0.88	22	3 750	1.97	400	455	36.4	491
2	" " " " "	6′ 0″	3.40	4.40	F.	0.24	4	3.06	0.99	22	3 550	1.31	436	431	39.6	471
3		6′ 0″	3.05	4.05	F.	0.27	4	2.75	0.89	22	3 900	1.72	401	473	36.5	510
6-ft. 1 yr.																
1	" " " " "	6′ 0″	3.21	4.21	F	0.26	4	2.92	0.87	20	4 138	1.50	425	502	38.6	541
2		6′ 0″	3.02	4.02	F.	0.27	4	2.75	0.82	20	3 800	1.61	406	461	36.9	501
5-ft. Am.																
1	" " " " "	5′ 0″	3.20	4.20	F.	0.26	4	2.88	0.99	26	3 875	0.72	346	567	37.6	605
2		5′ 0″	3.25	4.25	F.	0.25	4	2.93	1.01	26	3 050	0.44	351	443	38.2	481
5-ft. Am. 3-in																
1	" " " " "	5′ 0″	2.20	3.20	F.	0.38	4	1.93	0.79	26	950	0.82	264	138	28.7	167
2		5′ 0″	2.05	3.05	F.	0.40	4	1.80	0.74	26	2 215	0.81	252	322	27.4	349
5-ft. Ex.																
1	Expanded metal. Area 0.130 in.	5′ 0″	3.70	4.20	F.	0.15	4	3.40	0.89	26	2 690	0.55	346	391	37.6	429
2		5′ 0″	3.70	4.25	F.	0.15	4	3.40	0.89	26	3 005	0.33	351	437	38.2	475

TABLE 3.—(*Continued.*)

Slab No.	Reinforcement.	Span.	Thickness. Net.	Thickness. Gross.	End condition.	Percentage of steel.	Age, in weeks.	(jd)	(kd)	(u)	Maximum applied load, in pounds.	Maximum deflection, in inches.	Estimated weight, in pounds.	Equivalent load, in pounds per square foot.	Uniform dead load, in pounds per square foot.	Equivalent total uniform load, in pounds.
13–C–8																
1	Triangular wire mesh. Area, 0.175 in.	8′ 0″	3.10	4.10	R.	0.26	5	2.82	0.84	20	5 389	2.39	566	490	38.6	529
2		8′ 0″	3.07	4.07	R.	0.27	9	2.79	0.80	17	4 875	2.34	562	444	38.3	482
3		8′ 0″	3.05	4.05	R.	0.27	9	2.76	0.79	17	6 150	1.18	559	559	38.1	597
4		8′ 0″	3.00	4.00	R.	0.27	9	2.73	0.78	17	4 588	1.25	552	413	37.6	451
5		8′ 0″	3.00	4.00	R.	0.27	9	2.73	0.78	17	5 039	1.97	552	458	37.6	496
6		8′ 0″	2.97	3.97	R.	0.28	9	2.70	0.77	17	6 150	3.07	548	559	37.4	596
13–8																
1	"	8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 750	2.65	552	250	37.6	288
2	"	8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 970	4.28	552	270	37.6	308
3	"	8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 750	3.20	552	250	37.6	288
4	"	8′ 0″	3.12	4.12	F.	0.26	10	2.84	0.81	17	2 020	1.35	569	184	38.8	223
5	"	8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 520	5.48	552	229	37.6	267
6	"	8′ 0″	3.25	4.25	F.	0.25	10	2.96	0.85	17	2 520	3.40	587	229	40.0	269
13–8.5 in.																
1	"	8′ 0″	4.00	5.00	F.	0.21	10	3.68	0.92	17	3 470	5.22	691	316	47.2	363
2	"	8′ 0″	4.00	5.00	F.	0.21	10	3.68	0.92	17	3 720	5.91	691	339	47.2	386
3	"	8′ 0″	4.00	5.00	F.	0.21	10	3.68	0.92	17	3 470	4.05	691	316	47.2	363

TABLE 3.—(*Continued.*)

Slab No.	Reinforcement.	Span.	Thickness		End condition.	Percentage of steel.	Age, in weeks.	(jd)	(kd)	(n)	Maximum applied load, in pounds.	Maximum deflection, in inches.	Estimated weight, in pounds.	Equivalent load, in pounds per square foot.	Uniform dead load, in pounds per square foot.	Equivalent total uniform load, in pounds.
			Net.	Gross.												
5-ft. Tw.																
1	¼ square twisted. Area, 0.178	5′ 0″	3.10	4.10	F.	0.26	4	2.79	0.96	26	3 500	0.41	336	509	36.5	546
2	in.	5′ 0″	3.45	4.20	F.	0.24	4	3.10	1.07	26	3 590	1.62	346	522	37.6	560
5-ft. Sq.																
1	¼ square bars. Area, 0.202 in..	5′ 0″	3.27	4.27	F.	0.28	4	2.94	1.02	26	3 725	0.33	352	543	38.3	581
2		5′ 0″	3.25	4.25	F.	0.28	4	2.93	1.01	26	3 825	0.48	351	556	38.2	594
5-R.																
1	¼ round bars. Area, 0.153 in...	5′ 0″	3.20	4.20	F.	0.22	4	2.88	0.93	26	2 125	0.06	336	309	36.5	346
2		5′ 0″	3.25	4.25	F.	0.21	4	2.94	0.94	26	1 575	0.04	351	229	38.2	267
5-PL																
1	No reinforcement.....	5′ 0″	....	4.36	F.	....	4	....	...	..	1 500	0.019	360	219	39.3	258
2		5′ 0″	....	4.25	F.	....	4	....	....	..	1 510	0.027	351	220	38.4	258
12-C-6																
1		6′ 0″	3.22	4.22	R.	0.25	4	2.96	0.81	16	11 182	0.85	437	1 357	39.6	1 397
2		6′ 0″	3.22	4.22	R.	0.25	4	2.96	0.81	16	10 250	....	437	1 242	39.6	1 282
3	Triangular wire mesh. Area,	6′ 0″	3.00	4.00	R.	0.26	4	2.76	0.75	16	13 500	0.73	414	1 636	37.6	1 674
4	0.173 in.....	6′ 0″	3.37	4.37	R.	0.23	10	3.10	0.84	16	12 100	1.06	453	1 466	41 3	1 507
5		6′ 0″	3.00	4.00	R.	0.26	10	2.76	0.75	16	12 295	1.21	414	1 490	37.6	1 528
6		6′ 0″	3.10	4.10	R.	0.25	10	2.85	0.78	16	12 000	0.65	425	1 455	38.6	1 494
13-6																
1	" " " "	6′ 0″	3.10	4.10	F.	0.25	5	2.85	0.78	16	3 900	....	425	473	38.6	512
2		6′ 0″	3.10	4.10	F.	0.25	5	2.85	0.78	16	3 400	2.07	425	412	38.6	451
13-6																
1	Welded wire mesh Area, 0.238	6′ 0″	3.27	4.27	F.	0.33	5	2.98	0.92	16	3 250	2.35	442	394	40.1	434
2	in	6′ 0″	3.02	4.02	F.	0.36	5	2.75	0.85	16	3 900	3.00	416	473	37.8	511
3		6′ 0″	3.07	4.07	F.	0.35	5	2.80	0.86	16	3 150	1.98	421	382	38.2	420
13-6																
1		6′ 0″	3.05	4.05	F.	0.19	5	2.83	0.64	16	2 000	0.44	419	243	38.1	281
2	Expanded metal. Area, 0.130	6′ 0″	3.15	4.15	F.	0.18	5	2 93	0.66	16	2 100	0.63	430	254	39.1	293
3	in.....	6′ 0″	3.07	4.07	F.	0.18	5	2 86	0.65	16	2 250	1.05	421	273	38.3	311

for the thrust produced by the test span, was varied by each individual tester. Consequently, no standard practice was developed, and the test results obtained were not comparable. Frequently, the test span

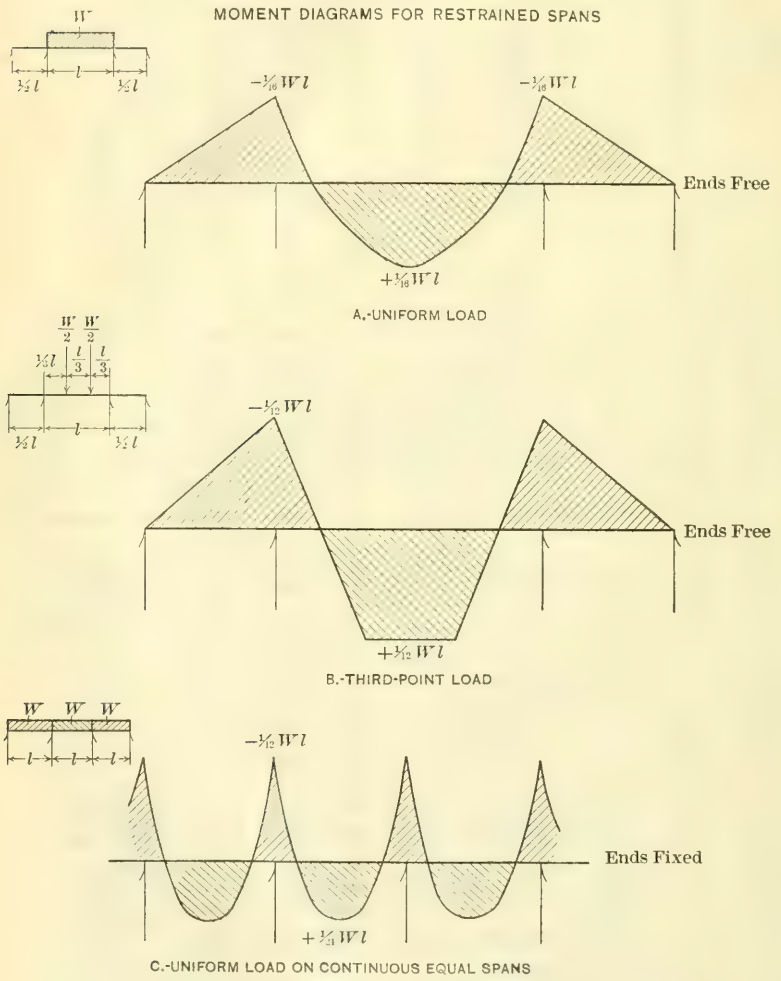


FIG. 20.

was rigidly held between two adjacent anchor-spans filled solidly with concrete, thus forming heavy haunches on both ends of the test span; and the reinforcement was securely anchored in these adjacent spans.

Observations of the tests led to the conclusion that, during the early stages of loading, the construction did not differ materially from the type of slab constructed continuously over two or more rigid supports. In planning the present series of tests, it was decided, therefore, to base the form of test specimen on the principles underlying the continuous type of beam. A comparison of the bending moments and shears for three types of loading are shown in Figs. 20 and 22 for the slab continuous over three spans, and in Fig. 21 for the simply supported slab. The factors for shears and moments in the former case were derived by the theorem of three moments, assuming full continuity of construction, fixed supports on the same

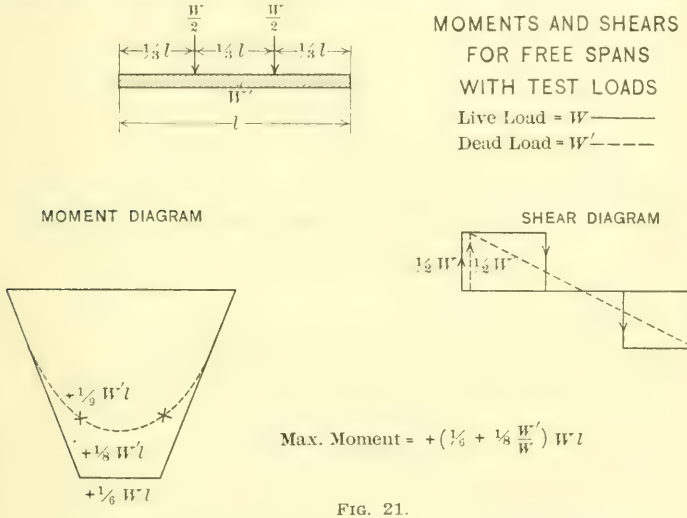


FIG. 21.

level, and constant moment of inertia of cross-section. It will be seen, from a study of these diagrams, that the conditions of shear and bending moment with the two-point loading closely approximate those obtaining with a uniformly applied load, with large shear occurring at the points of large negative bending moment. It will be observed, also, that the anchor-spans were in all cases assumed equal in length to one-half the test span, the purpose being to obtain the coefficients of bending moment ordinarily applied in the design of slabs continuous over two or more panels.

In all the tests on the simply supported and the restrained types of floors, the conditions of loading as outlined in Figs. 21 and 22

apply, respectively. The extreme ends of the specimens were in all cases considered to be freely supported, and the reinforcement was not secured at these points.

MOMENTS AND SHEARS FOR RESTRAINED SPANS WITH TEST LOADS

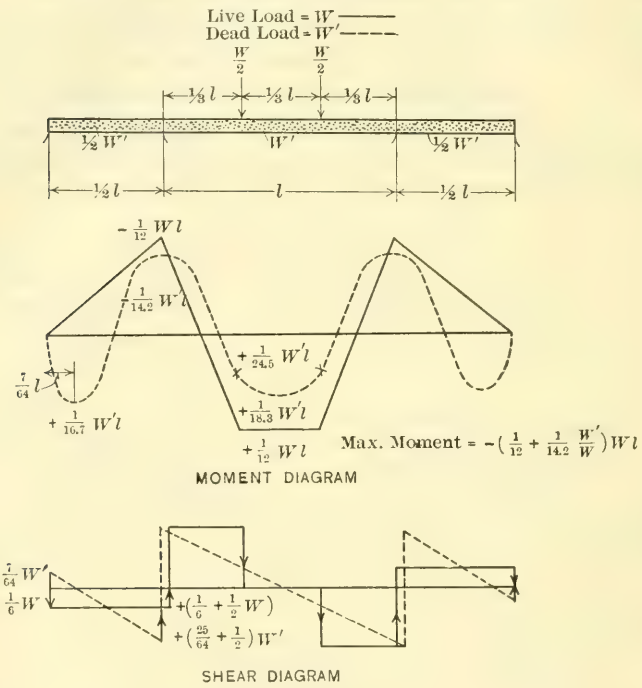


FIG. 22.

Let W = total live load, in pounds;

W' = weight of test span, in pounds;

l = length of test span, in inches;

M_s = moment at support, in inch-pounds;

M_c = moment at center, in inch-pounds;

and $M_{\frac{1}{3}}$ = moment at the third point, in inch-pounds.

Then, for the continuous or restrained construction,

$$M_s = + \left(\frac{1}{12} + \frac{1}{14.2} \times \frac{W'}{W}\right) W l \quad \ldots\ldots (1)$$

$$M_c = + \left(\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}\right) W l \quad \ldots\ldots (2)$$

$$M_{\frac{1}{3}} = + \left(\frac{1}{12} + \frac{1}{24.5} \times \frac{W'}{W}\right) W l \quad \ldots\ldots (3)$$

and, for the simply supported or free spans,

$$M_c = + \left(\frac{1}{6} - \frac{1}{8} \times \frac{W'}{W} \right) W\, l \ldots\ldots\ldots\ldots\ldots (4)$$

$$M_{\frac{1}{3}} = \left(\frac{1}{6} - \frac{1}{9} \times \frac{W'}{W} \right) W\, l \ldots\ldots\ldots\ldots\ldots (5)$$

It must be borne in mind that these formulas are to be applied only within the first stage of the test, as later described, which occurs within the region of working loads. With the third-point method of test, it is only necessary to be reasonably careful to maintain the load symmetrical with respect to the loading zones, as a 10% variation in one concentration as compared to the other will result in less than 4% variation in the applied bending moments.

The formula used in computing deflections is a modification of the form derived by Mr. G. A. Maney.*

Let y = deflection, in inches;
d = depth of slab to steel, from top of concrete, in inches;
e_c = unit deformation in the extreme fiber of the concrete, in inches;
e_s = unit deformation in steel reinforcement, in inches;
f_c = extreme fiber stress in concrete, in pounds per square inch;
f_s = unit stress in steel reinforcement, in pounds per square inch;
E_s = modulus of elasticity of steel;
E_c = modulus of elasticity of concrete;
and n = ratio $\frac{E_s}{E_c}$.

Then

$$y = \frac{k\, l^2}{d\, E_s} \times (n\, f_c + f_s) \ldots\ldots\ldots\ldots\ldots (6)$$

The constant, k, is the ratio of the quantity, k_1, in the expression for the deflection of a homogeneous beam, $k_1 \frac{W\, l^3}{E\, I}$ to the constant, k_2, in the expression for the maximum bending moments, $k_2\, W\, l$. The expressions for k_1 and k_2, with the loading under consideration, are as follows:

* *Proceedings*, Am. Soc. for Testing Materials, 1914.

For the continuous or restrained span, homogeneous beam,

$$y_1 = \left(\frac{1}{96} - \frac{1}{910} \times \frac{W'}{W}\right) \frac{W l^3}{E I} \quad \ldots\ldots\ldots (7)$$

or

$$k_1 = \left(\frac{1}{96} - \frac{1}{910} \times \frac{W'}{W}\right) \quad \ldots\ldots\ldots\ldots\ldots (8)$$

and, from Equation (2), $M_c = \left(\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}\right) W l$

or

$$k_2 = \left(\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}\right) \quad \ldots\ldots\ldots\ldots\ldots (9)$$

whence,

$$k_R = \frac{\frac{1}{96} - \frac{1}{910} \times \frac{W'}{W}}{\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}} \quad \ldots\ldots\ldots\ldots\ldots (10)$$

For the freely supported span, homogeneous beam,

$$y_2 = \left(\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}\right) \frac{W l^3}{E I} \quad \ldots\ldots\ldots (11)$$

or

$$k_1 = \left(\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}\right) \quad \ldots\ldots\ldots\ldots (12)$$

and, from Equation (4),

$$M_c = + \left(\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}\right) W l$$

or

$$k_2 = \left(\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}\right) \quad \ldots\ldots\ldots\ldots\ldots (13)$$

whence,

$$k_F = \frac{\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}}{\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}} \quad \ldots\ldots\ldots\ldots\ldots (14)$$

(The derivation of all the formulas used in this paper will be found in Appendix V.)

The moment and shear diagrams for the special type of test slab are shown by Fig. 23. This form of specimen was tested in order to determine the comparative strength of an end panel not provided with an adjacent anchor-span to take up the thrust of the test span. This condition is found in end panels adjacent to the exterior walls of a building and in bays adjacent to elevators and stair-wells.

Analysis of Slab Action.

On Figs. 32 to 38 are shown the curves expressing the relation between the load and the measured compressive and tensile strains.

with the corresponding observed deflections. It must be borne in mind that the compressive strain was observed at a plane $\frac{1}{2}$ in. down from the extreme fiber. To obtain the extreme fiber strain this measurement must be increased by the ratio of the distances from the neutral axis to the extreme fiber and to this point of measurement.

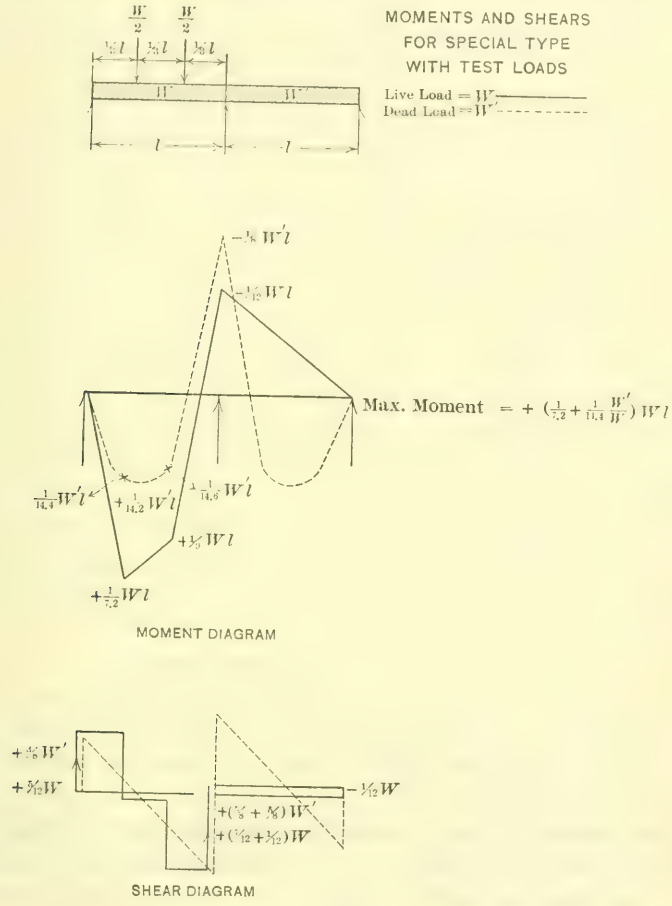


FIG. 23.

As noted on page 302, the tensile strain was measured on the exterior surface of the concrete at the plane of the steel reinforcement. It was to be expected, and subsequent results proved, that the true stretch of the steel could not be obtained in this manner, but the size of the reinforcing members and the thickness of this type of con-

struction prevent a more accurate treatment. Considering the shallowness of the slab and the small depth to the neutral axis, which was approximately 1 in., slight variations in the measurements, and the conversion of the observations into stresses, would lead to considerable error. Abnormalities are particularly to be expected in the steel stresses. However, in general, the curves of load and strain indicate the relations existing between the actual strains and deflections.

Simply Supported Slabs.—Analyses of the curves for the simple slabs show in general three stages of action. Up to a point on the curve designated as "first break", at a load corresponding to about one-third to one-half of the ultimate load on the slab, the ratio between load and stresses is direct. The observed strain in the steel is low, the deflection of the slab being very small. It is along this region of the curve that working loads occur. The "second stage" of the action is indicated by a rapid increase in the deflection, with a corresponding response in the working of the steel. Both tension in the steel and compression in the concrete increase rapidly up to a point corresponding to the yield point of the steel. What may be designated a third stage of the action occurs at this point, with a second break and a flattening out of the curve, running off rapidly to failure.

Restrained Slabs.—Analysis of the curves for the restrained slabs, built in between steel beams with reinforcement securely anchored, indicates a possible distribution of stresses differing from that to be expected from ordinary beam action.

The first stage again occurs with the "first break", at a load about twice as great as in the simply supported slab and about one-fourth to one-third of the ultimate load on the slab. Both steel and concrete stresses during this stage are low. The steel assumes its load very slowly, and the compression in the concrete is also lower than would be expected. These stresses may be influenced by either suspension or pure arch action. If there is pure suspension or catenary action, the measured concrete stresses would be less than the computed ones by the amount of this pure tension, and the steel stresses would be correspondingly large. If there is arch action, the steel stress would be low and the concrete compression would be distributed over a greater portion of the cross-section, giving possibly a lower unit intensity at the extreme fiber than would be indicated by the

ordinary theory of flexure. In the application of the ordinary theory of flexure, assumptions are made as to the relation between the concrete and steel stresses and the ultimate compressive value of concrete in flexure, about which there is little or no definite information. The flat slab held between rigid I-beam supports, with the resistance to lateral motion offered by adjoining panels, will develop considerable strength or end restraint, even without reinforcement. This is the so-called arch action referred to previously.

The second and third stages follow in general the action of the simple slab, except that the "curve breaks" for correspondingly reinforced slabs occur at absolute loads about twice as great as for the simple slab. The slab itself under load gives its first sign of failure in negative tension cracks over the haunches. Thereafter the adjoining anchor-spans deflect upward and the test span downward. Numerous tension cracks then appear along the bottom of the arch, developing most rapidly under one or both of the concentrations. Ultimate failure in all cases except two, as hereafter noted, occurred in tension in the steel. The percentage of steel was in all cases made purposely low, both to insure tension failure and because this type of construction in practice is generally used with a low percentage of reinforcement. It is possible that the concrete was stressed to its ultimate capacity preceding the failure in the reinforcement, but signs of crushing of the concrete were not visible to the eye until the tension break of the steel.

Conversion of Observations into Stresses.

During the first stage of action, in both types of construction, the straight-line variation of stresses has been assumed. The modulus of elasticity used has been taken from the stress-strain curves, Fig. 31, obtained from cylinders cast with concrete from the corresponding slabs. The absolute value of the modulus was taken as the average along the one-quarter to three-eighths points of the curve. It must be recognized that the modulus thus obtained is based on concrete under conditions of stress which do not represent those obtaining in the slab proper. As shown later, the modulus obtained from unreinforced slabs in flexure is considerably higher than that obtained from the cylinders.

The neutral axis was located and the factors given in Table 3 were derived on the assumptions usually made in applying the common theory of flexure for the composite material of concrete and steel. The ratio, n, was based on a modulus of elasticity of 30 000 000 lb. per sq. in. for steel. On Fig. 24 the symbols corresponding to the quantities given in the tables are indicated.

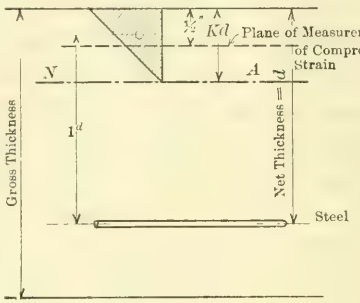


FIG. 24.

The tensile stress in the steel and the extreme fiber stress in compression are obtained in the following manner:

Let E_c = modulus of elasticity of the concrete, in pounds per square inch;

E_s = modulus of elasticity of the steel, in pounds per square inch;

n = ratio, $\frac{E_s}{E_c}$;

f_s = tension in steel, in pounds per square inch;

f_c = compression in extreme fiber, in pounds per square inch;

e_s = the measured tensile strain;

and e_c = the measured compressive strain.

Then
$$f_s = n \times e_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

and
$$f_c = n \times \frac{k\,d}{k\,d - \frac{1}{2}} \times e_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Computation of Derived Bending Moment Factors.

With the stresses in concrete and steel known, in addition to the total load applied on any one span of known thickness, the corresponding bending moment factor to be applied to produce these stresses can readily be ascertained. In Table 15 this factor has been derived in four different ways, based on the following quantities:

1.—Measured compression at known load;

2.—Measured tension at known load;

3.—Measured compression and deflection at known load; and

4.—Ratio of imposed loads on simple and restrained slabs.

The stress in the steel can be determined in two ways: First, from the observed deformation at the plane of the steel (Equation (15)); and second, from the observed deformation in the concrete and the observed deflection of the slab (Equation (6) and (16)).

Let w = load, in pounds per square foot at a point below the first break;

f_c = corresponding compressive stress measured;

f_s = corresponding steel stress measured;

y_1 = observed deflection, in inches;

f'_s = computed tension based on f_c and y_1;

k = ratio of depth of neutral axis to depth of steel;

j = ratio of effective depth to depth of steel;

L = length of span, in feet;

A_s = area of steel, in square inches;

M_r = resisting moment of section, $12 \frac{l}{c} w L^2 \times \frac{22}{12}$;

M_i = implied bending moment;

and $\frac{1}{c}$ = bending moment coefficient.

The width of slab is in all cases 22 in.

Method 1.—

$$M_r = 22 \times k d \times \frac{1}{2} f_c \times j d = 11 f_c k j d \ldots\ldots\ldots\ldots (17)$$

$$M_i = \frac{1}{c} \times w \times L \times \frac{22}{12} \times L \times 12 = \frac{22}{c} w L^2 \ldots\ldots\ldots\ldots (18)$$

And, from Equations (17) and (18),

$$C = \frac{2 w L^2}{f_c k j d^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Method 2.—

$$M_r = A \times T f_s \times j d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

$$M_i = \frac{22}{c} w L^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

And, from Equations (20) and (21),

$$C = \frac{22 w L^2}{A_s f_s j d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Method 3.—Assuming that the deformation measured in the exterior surface of the concrete in the plane of the steel reinforcement is

incorrect, and that the compressive deformation is more accurate, although it must be borne in mind that all such strain measurements are subject to various errors, then, from Equation (6), we have

$$y = \frac{k\,l^2}{d\,E_s}\,(n\,f_c + f_s').$$

Substituting for y the known deflection, and making also the proper substitution for the quantities, k, n, f_c, and E_s, the equation can be solved for f_s', the stress in the steel reinforcement, and from Equation 22, we have

$$C = \frac{22\,w\,L^2}{A_s\,f_s'\,j\,d} \qquad (23)$$

Method 4.—The fourth method of determining the implied bending moment coefficient is purely a rational one, and is based on a comparison of the ultimate loads supported by the simple and restrained types of specimens. These specimens were cast in groups of corresponding specifications, with both end conditions.

Let W_1 = equivalent uniform load, in pounds per square foot, supported by the simple slab;
and W_2 = equivalent uniform load, in pounds per square foot, supported by the restrained slab.

Then, since the bending moment coefficient for a simple slab loaded uniformly is $\frac{1}{8}$, the denominator of the coefficient for the restrained slab will be

$$C = 8\,\frac{W_2}{W_1} \qquad (24)$$

These values for the coefficient have been computed in Table 15, Appendix V, as well as the corresponding stresses computed by the ordinary straight-line theory of flexure, and the observed stresses in concrete and steel for a point just below the first break in the deformation curves of the 6 and 7-ft. free and restrained slabs.

It must again be cautioned that the attempt to measure fiber deformations in the surface of the concrete is at best only an approximate operation. In the case of thin slabs, such as those with which the present investigation is concerned, the depth to the neutral axis is so small that a slight variation in the location of the contact points

for measurement may produce a very large error. Moreover, the selection of the proper modulus of elasticity is an indeterminate feature. The method of determining safe loads and moment coefficients by the absolute supporting power of the slabs, as indicated by Method 4, is not subject to such errors, and presumably leads to approximately accurate results.

Modulus of Elasticity of Unreinforced Slabs.

Two concrete slabs, 22 in. wide and 4 in. thick, of the same mix as the 5-ft. reinforced specimens, were tested on a simple span, and the deflections of the slab were accurately measured. Then from the common theory of flexure, using Equation (12), we have

$$y = y' + y' \left(\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W} \right) \frac{W l^3}{E I}.$$

Substituting the value of y, and W, W', e, and I, the equation can be solved for E.

The values of the coefficient of elasticity thus obtained have been given in Table 16. It will be noted that these values are higher than those obtained for the cylinders in compression. Inasmuch as the tensile value of concrete is ordinarily neglected in the design of reinforced concrete beams and slabs in flexure, it is customary to assume a smaller value of the coefficient of elasticity to compensate for this neglect, which is equivalent to assuming a larger ratio of E_s to E_c, thus lowering the apparent location of the neutral axis in cases of small percentages of steel or over-design in concrete. In the conversion of strains to stresses in this investigation, the ratio, n, has been obtained from the actual coefficients of elasticity as determined from the cylinder tests, as this in itself is possibly lower in value than would be obtained from concrete under the conditions of stress existing in the slab.

Derivation of Empirical Formulas.

In the ordinary practice of cinder concrete floor construction between steel beams, as applied in New York City, the safe loads to be supported by the construction have been determined by load tests of various characters, as described in the early part of this paper. During the past two years, it has been suggested, by a joint committee of architects and engineers, that the results of the more satisfactory of these tests be combined in a table from which the safe load

for varying conditions of span and reinforcement can readily be obtained. Such a table was prepared and has been embodied in two of the drafts of a proposed code for New York City.

A study of the results of this investigation would indicate that this procedure is rather drastic, and that the working loads thus obtained are excessive in some cases and discriminating in others. An attempt was made, therefore, to develop empirical formulas which would check the theoretical design of floors of this type when based on the coefficients developed by the investigation.

Let $w_1 = \frac{1}{8}$ of the equivalent ultimate uniform load on the slab at failure, in pounds per square foot;

and w_2 = the dead load or weight of the slab, in pounds per square foot.

Then the safe total load was assumed as $w_1 + w_2$. These safe loads are given in Table 4. A study of the influence of variations in area of steel reinforcement, thickness of slab, length of span, and conditions of end restraint was made with a view of determining the relative weights of these variables in fixing the safe load.

Let a = area of steel, in square inches;

l = length of span, in inches;

d = depth of steel from extreme upper fiber, in inches;

and W_t = total safe load, in pounds per square foot.

It was found that, for the restrained slabs,

W_t is proportional to $a^{0.84}$ ..... (1)

W_t is proportional to $d^{0.99}$ ..... (2)

W_t is proportional to $\frac{1}{l^{2.5}}$ ..... (3)

The assumption was then made, therefore, that

$$W_t = K_1 \frac{d\,a}{l^2} \quad \ldots\ldots (25)$$

This equation has the general form of the equation for the load on a suspension cable, expressed in terms of the horizontal tension in the lowest point of the cable and the sag or depth, that is

$$H = \frac{\frac{1}{8} w\, l^2}{d} = a\, s, \text{ or } w = 8\, S \frac{d\,a}{l^2}.$$

Substituting the corresponding values for W_t obtained from the tests in Equation (25), the following values were obtained for K_1.

*Official, 7 ft. 6 in.	4 300 000
*Official, 8 ft. 0 in.	2 800 000
C — 7 ft. 1 in.	3 600 000
C — 6 ft. 0 in.—1914	3 400 000
C — 6 ft. 0 in.—1913	3 940 000
C — 8 ft. 0 in.—1913	3 150 000

From the foregoing the average value of K is approximately 3 500 000, or, for the restrained slabs,

$$W_t, \text{ in pounds per square foot, } = 3\ 500\ 000\ \frac{d\ a}{l^2} \quad \ldots\ldots\ldots (26)$$

In the case of plain bars hooked over the steel beams, without lateral reinforcement, Equation (26) does not apply. A study of all the available data would indicate that the following expression meets the conditions of that type of reinforcement:

$$W_t, \text{ in pounds per square foot, } = 2\ 500\ 000\ \frac{d\ a}{l^2}.$$

In the same manner, the safe total loads for the simple slabs were found, and the weight of each factor was determined. The following proportions were developed:

W_t is proportional to $a^{0.75}$ ..... (1)

W_t is proportional to $d^{1.3}$ ..... (2)

W_t is proportional to $\frac{1}{l^{1.3}}$ ..... (3)

It was assumed, therefore, that

$$W = K_2\ \frac{d^2\ a}{l^2} \quad \ldots\ldots\ldots (27)$$

With this formula as a basis, the solution gave the following values of K_2:

	7 ft. 1 in.—1914	600 000
	6 ft. 0 in.—1914	565 000
A — 4 in.	— 5 ft. 0 in.—1914	425 000
A — 3 in.	— 5 ft. 0 in.—1914	540 000
Ex.	5 ft. 0 in.—1914	840 000

* These data were obtained from the records of the Bureau of Buildings, from tests made under conditions practically identical with those of the investigation.

Tw.		5 ft. 0 in.—1914	415 000
Sq.		5 ft. 0 in.—1914	430 000*
Rd.		5 ft. 0 in.—1914	320 000*
A		6 ft. 0 in.—1913	555 000
Cl.		6 ft. 0 in.—1913	390 000
Ex.		6 ft. 0 in.—1913	550 000
A	4 in.	8 ft. 0 in.—1913	710 000
A	5 in.	8 ft. 0 in.—1913	520 000

The average value of the foregoing is approximately 550 000, whence, for the simple slabs unrestrained by steel beams and without anchored reinforcement,

$$W_t, \text{ in pounds per square foot, } = 550\,000 \frac{d^2}{l^2} a \quad \ldots\ldots\ldots (28)$$

Bond Failure.—Four of the simple slabs of 5 ft. 0 in. spans, reinforced respectively with $\frac{1}{4}$-in. square and $\frac{1}{4}$-in. round rods spaced 8 in. apart on centers, failed by slipping of the bond of concrete and steel.

Let U = bond, in pounds per square inch;

ΣO = total circumference of the rods, in inches;

V = end reaction, in pounds.

Then

$$U = \frac{V}{\Sigma O\, jd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Computing the bond stress at failure by Equation (29), it was found that, at failure, U was 184 lb. per sq. in. for round rods and 255 lb. per sq. in. for square rods. On the other hand, the twisted square reinforcement which failed in tension was subject to a bond stress of 250 lb. per sq. in. Although these tests are not conclusive enough, the general indication is that a safe bond stress of about 40 lb. per sq. in. should not be exceeded in the case of unanchored or loose reinforcement.

In Table 4 have been compiled the safe loads as determined by test and the loads obtained from Equations (26) and (28). From a study of the typical cinder concrete gathered from about the city (Table 1), and the concrete used in the test slabs (Table 8), the following working units were selected as being best suited for the basis of design:

Extreme fiber stress in concrete.... 300 lb. per sq. in.

Tensile stress in steel.....16 000 " " " "

* Bond failures, omit in averages.

TABLE 4.—Safe Loads, Based on Test and Empirical Formulas.

Restrained Type, $w = 3\,500\,000 \frac{d\,a}{l^2}$. Simple Type, $w = 550\,000 \frac{d^2\,a}{l^2}$.

w = pounds per square foot, total load;
d = depth to steel, in inches;
a = area of steel per foot of slab, in square inches;
l = length of span, in inches.

Type.	Safe Load. One-eighth of test load plus dead load. (2)	Safe Load. Empirical formula. (3)	Ratio of formula load to ultimate load. (4)	Coefficient, $\frac{1}{C}$, Based on: Ratio, $n = 30$ and $f_1 = 300$ and Load, Column (2). Concrete. (5)	Steel. (6)	Load, Column (3). Concrete. (7)	Steel. (8)	Ratio, n, Determined by Test and Load, Column (3). Concrete. (9)	Steel. (10)
C—7 ft. 1 in.	140	137	6.4	$\frac{1}{23.6}$	$\frac{1}{27.4}$	$\frac{1}{23.4}$	$\frac{1}{27.3}$	$\frac{1}{16.8}$	$\frac{1}{19.5}$
C—6 ft. 0 in., 1914	185	182	6.8	$\frac{1}{23.4}$	$\frac{1}{27.1}$	$\frac{1}{23.1}$	$\frac{1}{26.7}$	$\frac{1}{18.2}$	$\frac{1}{19.2}$
C—6 ft. 0 in., 1913	216	182	8.0	$\frac{1}{26.4}$	$\frac{1}{31.4}$	$\frac{1}{22}$	$\frac{1}{26.4}$	$\frac{1}{18.2}$	$\frac{1}{19.2}$
C—8 ft. 0 in., 1913	97	107	4.9	$\frac{1}{22.2}$	$\frac{1}{25.7}$	$\frac{1}{24.3}$	$\frac{1}{28.4}$	$\frac{1}{21}$	$\frac{1}{20}$
Official, 7 ft. 6 in.	274	225	7.7	$\frac{1}{17.5}$	$\frac{1}{28.0}$	$\frac{1}{15.1}$	$\frac{1}{23.0}$	$\frac{1}{11.2}$†	$\frac{1}{19.8}$
Official, 8 ft. 0 in.	172	214	5.3	$\frac{1}{32.0}$	$\frac{1}{25.5}$	$\frac{1}{39.8}$	$\frac{1}{31.6}$	$\frac{1}{28}$	$\frac{1}{21.2}$
Triangular, 7 ft. 1 in.	70	65	5.0	$\frac{1}{10.0}$	$\frac{1}{11.8}$	$\frac{1}{9.8}$	$\frac{1}{11.7}$	$\frac{1}{7.2}$	$\frac{1}{8.6}$
Triangular, 6 ft. 0 in., 1914	92	90	5.5	$\frac{1}{11.1}$	$\frac{1}{13.3}$	$\frac{1}{10.9}$	$\frac{1}{13.1}$	$\frac{1}{8.2}$	$\frac{1}{9.1}$
Triangular, 4 in., 8 ft. 0 in., 1913.	65	51	5.4	$\frac{1}{14.9}$	$\frac{1}{17.3}$	$\frac{1}{11.6}$	$\frac{1}{13.6}$	$\frac{1}{9.9}$	$\frac{1}{9.4}$
Triangular, 5 in., 8 ft. 0 in., 1913.	85	90	4.1	$\frac{1}{12.0}$	$\frac{1}{16.3}$	$\frac{1}{12.6}$	$\frac{1}{17.3}$	$\frac{1}{11.3}$	$\frac{1}{12.5}$
Welded, 6 ft. 0 in., 1913	88	124	3.7	$\frac{1}{9.8}$	$\frac{1}{9.6}$	$\frac{1}{13.8}$	$\frac{1}{13.6}$	$\frac{1}{12.5}$	$\frac{1}{9}$
Exp., 6 ft. 0 in., 1913.	68	97	3.05	$\frac{1}{9.7}$	$\frac{1}{14.7}$	$\frac{1}{13.9}$	$\frac{1}{20.3}$	$\frac{1}{12.5}$	$\frac{1}{12.9}$
Triangular, 6 ft. 0 in., 1913	90	90	5.3	$\frac{1}{11.3}$	$\frac{1}{13.5}$	$\frac{1}{11.3}$	$\frac{1}{13.5}$	$\frac{1}{9.7}$	$\frac{1}{9.1}$
4 in., 5 ft. 0 in., 1914.	100	129	4.2	$\frac{1}{8.2}$	$\frac{1}{9.5}$	$\frac{1}{10.5}$	$\frac{1}{12.3}$	$\frac{1}{7.4}$	$\frac{1}{8.9}$
3 in., 5 ft. 0 in., 1914.	56	57	4.5	$\frac{1}{9.1}$	$\frac{1}{8.5}$	$\frac{1}{9.1}$	$\frac{1}{8.5}$	$\frac{1}{6.7}$	$\frac{1}{6.1}$
Exp., 5 ft. 0 in., 1914.	88	98	4.6	$\frac{1}{6.7}$	$\frac{1}{10.6}$	$\frac{1}{7.4}$	$\frac{1}{11.7}$	$\frac{1}{5.4}$	$\frac{1}{7.7}$
Tw., 5 ft. 0 in., 1914.	100	133	4.2	$\frac{1}{7.7}$	$\frac{1}{9.2}$	$\frac{1}{10.3}$	$\frac{1}{12.2}$	$\frac{1}{8.7}$	$\frac{1}{7.5}$
Sq., 5 ft. 0 in., 1914*.	105	150	3.9	$\frac{1}{7.9}$	$\frac{1}{8.7}$	$\frac{1}{11.3}$	$\frac{1}{12.4}$	$\frac{1}{8.4}$	$\frac{1}{8.7}$
Rd., 5 ft. 0 in., 1914*.	70	115	2.7	$\frac{1}{6}$	$\frac{1}{8.2}$	$\frac{1}{9.8}$	$\frac{1}{13.4}$	$\frac{1}{7.1}$	$\frac{1}{8.8}$

* Failure in bond.

† This slab had 1 : 2 cement finish 1 in. thick; an extreme fiber stress of 600 lb. per sq. in., which makes $\frac{1}{C} = \frac{1}{24}$.

With these factors as a basis of computation, using the value of the ratio, n, as determined by test on the slab concrete in Columns 9 and 10, and a ratio of 30 in Columns 5, 6, 7, and 8, of Table 4, the corresponding factor for implied bending moments was derived, as shown in that table. The factor in the bending moment formula, $\frac{1}{C} W l$, for the simple slabs, was found to be approximately $\frac{1}{8}$ as expected, and for the restrained slabs approximately $\frac{1}{20}$. (See Columns 9 and 10 of Table 4.)

No special care was taken in preparing the concrete for these tests, nor was the material specially selected. It is to be presumed, therefore, that the results are indicative of what may be expected from cinder concrete placed with ordinary care and under proper supervision. With the foregoing loads considered as safe, the average factor of safety on the ultimate load basis is approximately 5.

Acknowledgments.

In addition to those already mentioned, the writers wish to make the following acknowledgments: The tests were conducted by the Department of Civil Engineering, Columbia University, partly with the co-operation of the Bureau of Buildings, Borough of Manhattan, and were made possible through the action of William H. Burr, M. Am. Soc. C. E., who made available the William Richmond Peters, Jr., Fund for Engineering Research. The Bureau was represented in the field by K. M. Boorman, Engineer-Inspector. The actual tests were run by Messrs. F. Miller and J. P. Maloy, Assistants in the Department of Civil Engineering, Columbia University, and, in addition, in 1913, by E. H. Kirchgraber, Jun. Am. Soc. C. E., and Mr. G. Lommel, and, in 1914, by Messrs. S. Quattlebaum, G. Richardson, and H. C. Stevens, all senior students in Civil Engineering at Columbia University.

Conclusions and Recommendations.

From general experience with anthracite clinkers, and knowledge gained from the present investigation, the following conclusions may be drawn, and on these the writers have based their recommendations.

As to Fire Resistance.—

1.—Cinder concrete, even though cast in a lean mixture and of low-grade material, is an extremely effective fire-resisting product.

2.—Steel, especially if of small cross-section and high carbon content, placed within 1 in. of the exposed surface is apt to be annealed to a marked extent, with consequent loss in strength, when the embedding concrete is subjected to fire, under conditions obtaining in conflagrations. This observation, however, might easily be made with reference to concrete mixed from any common aggregate.

3.—Under ordinary conditions, the pitting which occurs to a depth probably not greater than 1 in., even in extreme cases, and is caused by the combustion of particles of exposed coal and by the effect of de-hydration, may be readily repaired by plastering directly on the damaged surface, without the necessity of lathing or other means of support.

As to Corrosive Effects.—

1.—Anthracite cinder concrete (1:2:5), well mixed, cast in a viscous to wet consistency, constantly stirred and mixed up during placement, in such a manner as to coat the reinforcement thoroughly with mortar, will not cause the corrosion of embedded steel.

Table 5 shows the chemical constituents of the cinders, as determined by analysis. The first four samples represent the cinders used in the concretes reported as typical New York material. The last two samples represent the aggregate used in the test slab concrete.

TABLE 5.—Chemical Constituents of Cinders.

Samples.	Percentage of unburned coal determined by carbon loss on ignition.	Percentage of sulphides.	Percentage of sulphuric anhydride. (SO_3)
A	16.50	0.073	0.21
B_1	28.10	0.091	0.14
B_2	25.75	0.084	0.08
C	26.41	0.107	0.07
Slabs, 1913	15.96	0.063	0.09
Slabs, 1914	10.84	0.056	0.03

It will be observed that the quantity of sulphur is in general small and increases with the unburned coal content. The sulphur existing as sulphides is probably inert. The sulphuric anhydride which is present in the cinders as sulphates may be an agent of corrosion, in the following manner. If in the form of iron sulphate, chemical action may result in the presence of the cement and moisture, resulting in the formation of calcium sulphate and iron oxide. Professor

Norton has shown that incipient corrosion has occurred where iron oxide is in contact with the steel. The only safeguard is to insure the presence of a protective coating of cement mortar on the steel reinforcement, by the use of a mixture of a viscous or wet consistency.

As to the Characteristics of the Typical New York Product.—

1.—The variableness of New York cinder concrete is marked. Other conditions being identical, a difference of 150% in compressive strength might be expected to be shown by two series of specimens, due alone to the quality of the cinders.

2.—Hand-mixed cinder concrete handled in the ordinary manner is inferior to the machine-mixed product.

3.—The average weight of reinforced cinder concrete, based on actual investigation, was found to be 108 lb. per cu. ft. It is recommended that this be the minimum value assumed in design.

4.—The crushing strength of standard cylinders of good cinder concrete as used in New York is between 800 and 1 000 lb. per sq. in. at 28 days. Poor material will give results as low as 400 lb. per sq. in.

5.—The crushing strength increases for a period of about 6 months, after which it remains practically constant for a period of a year.

6.—The modulus of elasticity, E, under working loads, determined from standard cylinders, is about 1 200 000 lb. per sq. in. for average concrete at 28 days. It varies only slightly with age.

As to Proportions and Mixing in General.—

1.—The assumption that screened cinders will produce better concrete was investigated, with the result that 1 : 2 : 5 concrete, made with cinders from which all material less than $\frac{1}{4}$ in. diameter had been removed, showed a decrease in compressive strength of more that 25 per cent. (See Appendix IV.)

2.—Cinder concrete should be mechanically-mixed, for not less than 2 min., not leaner than 1 : 2 : 5, giving a minimum compressive strength, with standard cylinders at 28 days, of 800 lb. per sq. in., and be in a wet to viscous consistency when cast. It should be made from hard, well-burned, vitreous clinkers, free from fine, powdery ash. It has been found from actual test that unburned coal, existing to the extent of from 15 to 25%, is not necessarily detrimental to the strength or fire-resisting value of such concrete.

3.—Large irregular clinkers, often 2, 3, or 4 in. or even larger, in diameter, should be thoroughly broken up, by shovels or otherwise, before mixing, as they are sources of weakness in the finished product, if allowed to remain intact. This is due, not only to the inherent weakness of the aggregate itself, but also to the difficulty with which the mortar may reach all parts of its porous surface, a condition necessary for maximum density.

As to Future Regulation.—

1.—The safe load of cinder concrete floor fillings should be determined by any one of the following methods:

A.—Design.

a.—Common Theory of Flexure.

For slabs restrained between steel beams, with reinforcement thoroughly anchored, such as continuous mesh with proper cross-wires:

Bending moment coefficient.....	$\frac{1}{20}$
Extreme fiber stress in concrete..	300 lb. per sq. in
Steel stress 16 000	" " " "
Ratio, n	30

For slabs simply supported with non-continuous reinforcement:

Bending moment coefficient.....	$\frac{1}{8}$
Extreme fiber stress.....	300 lb. per sq. in.
Steel stress 16 000	" " " "
Ratio, n	30
Bond stress	40 lb. per sq. in.

Note that the safe bond stress of 40 lb. per sq. in. should be applied to loose bar reinforcement. In the case of mesh, the cross-wires give sufficient mechanical bond.

b.—Empirical Formula.

Restrained slabs, total safe load per square foot $= 3\,500\,000 \frac{d\,a}{l^2}$. To be applied to continuous mesh with proper cross-wires.

Restrained slabs, total safe load $= 2\,500\,000 \frac{d\,a}{l^2}$. To be applied to longitudinal bars hooked over the beams.

Simple slabs, total safe load $= 550\,000 \frac{d^2\,a}{l^2}$.

In the application of this formula, investigation should be made of the bond stress when loose bars are used.

Where d = depth of steel, in inches;
a = area of longitudinal steel per foot of width of slab, in square inches;
l = length of span, in inches;
w = load, in pounds per square foot, including the weight of slab.

The span should be limited to 8 ft. in the application of the special formulas given herein, and the minimum total thickness to one-eighteenth of the span length. It should also be noted that the empirical formulas are based on tests of slabs reinforced with steel varying from $\frac{1}{4}$ to $\frac{3}{8}$ per cent. For the stresses assumed in this analysis, the critical percentage of steel is about 32 per cent. (Table 17.)

For end-span conditions, the safe load, as determined by the formulas, should be reduced to three-fifths of the foregoing recommended values; or the strength of the construction should be increased proportionately.

B.—Load Test.

The load test should be conducted on a section of floor system not less than 4 ft. wide and with a span between steel beams equal to the span for which approval is desired. This specimen should be constructed as in actual practice, with material of the same quality as that incorporated in the specimen previously subjected to the standard fire, load, and water tests, and which is thereafter to be used in the system. In all cases the specimen should consist of a test span of normal dimensions and two anchor-spans, each of one-half the length of the test span. These anchors should conform to the construction of the test bay in every detail except in length.

In the case of reinforcement manufactured in sheets, which is to be put in with laps between sheets, the lap must come within the middle third of the test span.

The specimen should be tested either by stacking pig iron, bags of sand, or any other available material of sufficient specific gravity, on a platform supported on timbers placed so that the test load will be applied at two points, or loading areas, at the one-third points of the test span. Each loading area should consist of a zone of the

slab surface not exceeding 1 ft. in width and of a length conforming to the width of the test slab. A loose sand bed may be provided, not more than 1 in. thick, at each loading area or zone; or the specimen should be loaded similarly in a testing machine, by a calibrated hydraulic jack, or by other mechanical means. The construction should be tested at the age of 1 month.

It is recommended that approval be granted for a live load per square foot of not more than one-eighth of the equivalent uniform imposed load necessary to cause the failure of the slab. A factor of safety of 8, on the live load basis, is equivalent to approximately 6 on the total load.

APPENDIX I

Typical New York Material.

Curves and Additional Data.

TABLE 6.—Mechanical Analysis of Sands and Cinders Used in Casting Typical New York Product.

Zone.	Mix.	Material.	Percentage Passing Sieve.									
			1¼-in.	1-in.	¾-in.	½-in.	¼-in.	No. 10.	No. 30.	No. 40.	No. 50.	No. 100.
A	1 : 2 : 5	Sand.....	.....	.....	.....	100.0	98.8	88.9	58.5	39.5	29.3	7.3
		Cinders.....	97.8	97.2	93.4	85.9	63.4	38.8	24.4	22.8	21.6	13.7
B_1	1 : 1 : 5	Sand.....	.....	.....	.....	100.0	96.4	79.0	62.9	41.4	30.3	7.6
		Cinders.....	100.	98.2	93.4	76.1	34.1	11.1	5.0	3.7	3.5	1.4
B_2	1 : 2 : 5	Sand.....	.....	.....	.....	100.0	98.3	75.9	56.3	33.4	25.3	6.5
		Cinders.....	94.4	.....	80.3	57.3	24.3	11.7	9.3	6.7	6.2	3.6
C	1 : 2 : 5	Sand.....	.....	.....	.....	100.0	99.2	80.8	57.0	38.3	25.8	4.2
		Cinders.....	98.1	96.3	88.3	64.5	25.4	11.1	3.0	1.5	1.3	0.5

TABLE 7.—Tests on Cements Used in Casting Typical New York Product.

Zone.	Brand.	Specific gravity.	Percentage Finer than Sieves:		Percentage of water.	Tensile Strength:				
			No. 100.	No. 200.		24 hours.	7 days.	28 days.	56 days.	1 year.
A	Alsen.....	3.180	94.5	82.2	22½	308	566	695	713	725
B_1	Dragon.....	3.175	96.7	82.9	23	328	612	566	670	653
B_2	Vulcanite....	3.190	96.4	84.7	23	209	...	648	702	653
C	Atlas.....	3.150	97.7	81.2	22½	301	476	602	638	622

TABLE 8.—Typical Cylinder Test.

C —1 Month—No. 4.

Diameter = 8.07 in. Weight = 50 lb. 4 oz.

Height = 16.00 in. Area = 51.1 sq. in.

Total load, in pounds.	Unit stress, in pounds per square inch.	Unit deformation, in inches.
500	10	0.000000
2 500	49	0.000030
4 500	88	0.000075
6 500	127	0.000117
8 500	166	0.000150
10 500	205	0.000183
12 500	245	0.000233
14 500	284	0.000333
17 500	342	0.000375
20 500	401	0.000466
23 500	460	0.000573
26 500	519	0.000725
29 500	578	0.000800
32 500	636	0.000960
35 500	695	0.001185
41 100	804 Ultimate strength, in pounds per square inch.	

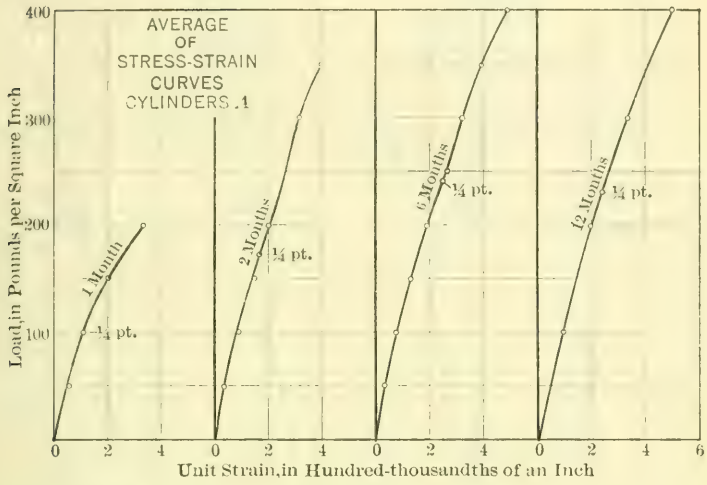

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS .1
1 Month
¼ pt.
2 Months
¼ pt.
6 Months
¼ pt.
12 Months
¼ pt.
Load, in Pounds per Square Inch
400
300
200
100
0
2
4
6
Unit Strain, in Hundred-thousandths of an Inch

FIG. 25.

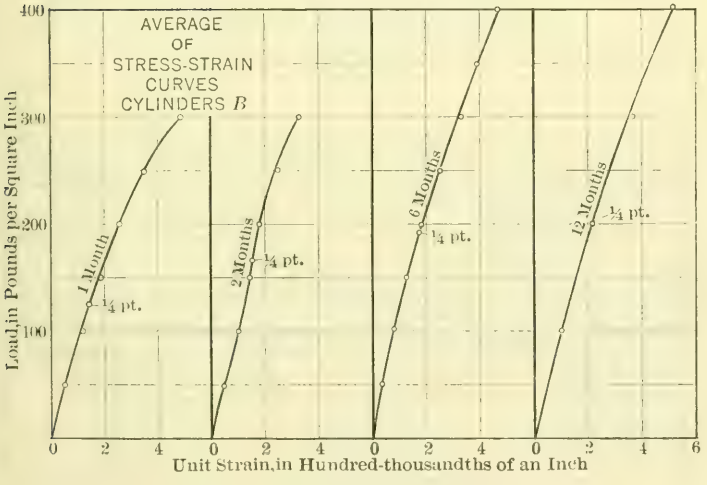

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS B
1 Month
¼ pt.
2 Months
¼ pt.
6 Months
¼ pt.
12 Months
¼ pt.
Load, in Pounds per Square Inch
400
300
200
100
0
2
4
6
Unit Strain, in Hundred-thousandths of an Inch

FIG. 26.

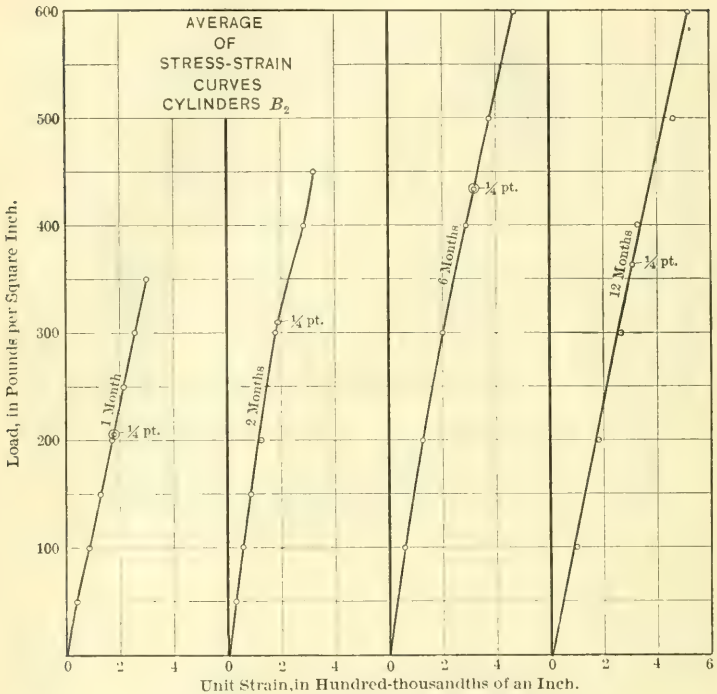

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS B_2
Load, in Pounds per Square Inch.
600
500
400
300
200
100
1 Month
2 Months
6 Months
12 Months
¼ pt.
0
2
4
6
Unit Strain, in Hundred-thousandths of an Inch.

FIG. 27.

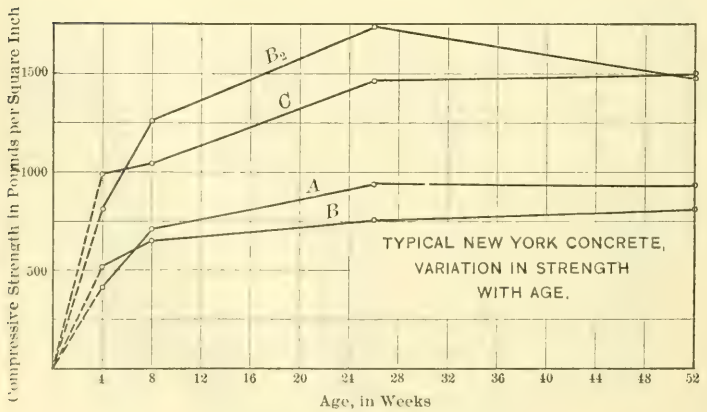

Compressive Strength in Pounds per Square Inch
1500
1000
500
B_2
C
A
B
TYPICAL NEW YORK CONCRETE,
VARIATION IN STRENGTH
WITH AGE.
4
8
12
16
20
24
28
32
36
40
44
48
52
Age, in Weeks

FIG. 28.

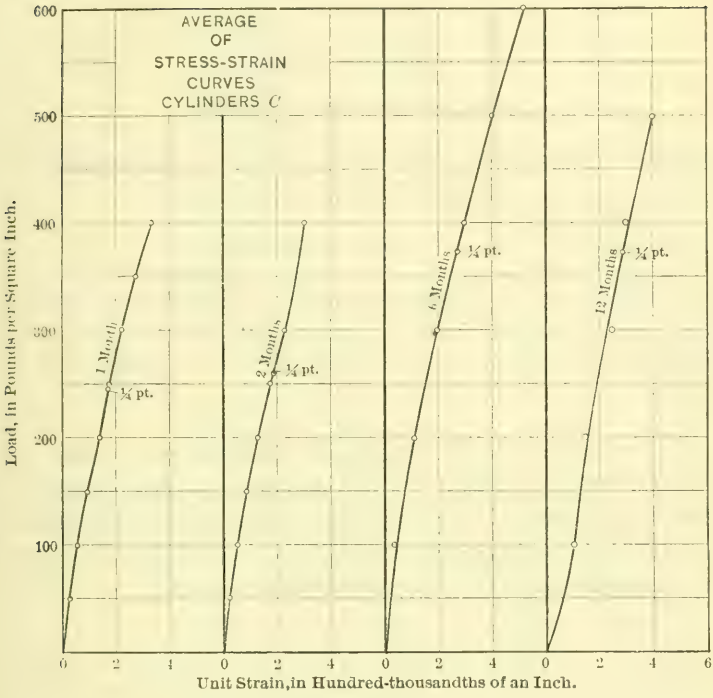

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS C
1 Month
¼ pt.
2 Months
¼ pt.
6 Months
¼ pt.
12 Months
¼ pt.
Load, in Pounds per Square Inch.
600
500
400
300
200
100
0
2
4
6
Unit Strain, in Hundred-thousandths of an Inch.

FIG. 29.

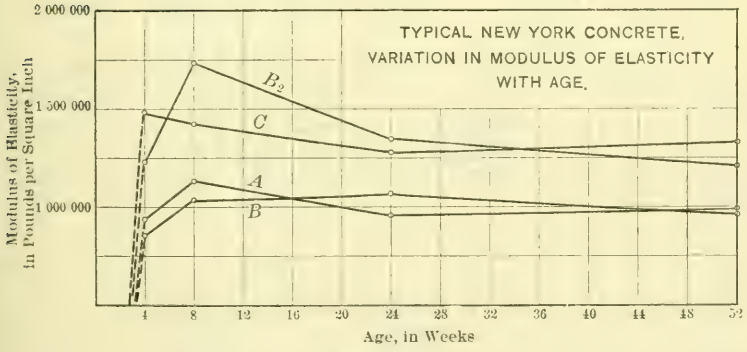

TYPICAL NEW YORK CONCRETE,
VARIATION IN MODULUS OF ELASTICITY
WITH AGE.
B_2
C
A
B
Modulus of Elasticity,
in Pounds per Square Inch
2 000 000
1 500 000
1 000 000
4
8
12
16
20
24
28
32
36
40
44
48
52
Age, in Weeks

FIG. 30.

APPENDIX II

Typical Slabs.

Data on Component Materials.

TABLE 9.—Mechanical Analyses of Sand and Cinders Used in Slab Tests, 1914.

	Percentage Passing Sieve.										
	1½-in.	1-in.	¾-in.	½-in.	¼-in.	No. 10.	No. 16.	No. 30.	No. 40.	No. 50.	No. 100.
Sand	.....	.....	.....	.....	100	89.0	78.8	48.3	29.7	15.5	2.25
Cinders	100	94.3	90.3	80.5	73.9	15.1	9.5	5.1	4.2	3.4	2.3

All determinations of mechanical analysis were made with a standard "rocker", down to ¼ in., and, for less than this size, with a Riehlé sand shaker. In the operation of the former the standard was assumed to be 100 complete cycles; and, in the latter, the standard was assumed to be 200 revolutions of the fly-wheel.

TABLE 10.—Tests on Cement Used in Slab Tests, 1914.

Brand: Dragon	Percentage Passing No. 100 Sieve: 96.3
Specific Gravity: 3.13	" " " 200 " 83.4
Soundness: O. K.	Normal Consistency, 25.0 per cent.

Tensile Strength, in Pounds per Square Inch.

Period.	24 hours.	7 days.	28 days.	56 days.
Neat	202	643	702	616
Sand	. . .	199	328	352

Compressive Strength, in Pounds Per Square Inch, on 2-In. Cubes.

Neat	1 269	5 606	6 132	6 952
Sand	....	1 468	1 445	1 521

TABLE 11.—Tensile Tests on Reinforcement.

	1913 Series.			1914 Series.					
	Cold-drawn wire			Cold-drawn wire, triangular mesh.		Rods.			
	6-ft. spans, triangular mesh.	6-ft. spans, welded mesh.	8 ft. spans, triangular mesh.	7-ft. spans.	5 and 6-ft. spans.	Square.	Round.	Twisted.	Expanded metal.
Area, in square inches	0.0293	0.0338	0.0288	0.0287	0.0288	0.0662	0.0510	0.0593	0.0099
Percentage of reduction in area	.....	.....	.....	45.2	56.6	55.1	73.8	66.9	.....
Percentage of elongation in 8 in	.....	.....	.....	5.6	4.6	3.6	22.9	8.5	.....
Yield point, in pounds per square inch	80 500	70 670	74 500	80 400	82 615	66 330	34 575	50 630	.....
Ultimate strength, in pounds per square inch	97 440	77 220	85 870	103 600	104 470	77 200	50 950	62 930	62 600

TABLE 12.—Crushing Strength, etc., of Slab Concrete.

	Material from Furnald Hall, Columbia University, Used in 1913 Slabs.		Material Used in 1914 Slabs, Cast in Laboratory.		
	6-ft. slabs.	8-ft. slabs.	5-ft. slabs.	6-ft. slabs.	7-ft. slabs.
Mix	1 : 2 : 5	1 : 2 : 5	1 : 2 : 5	1 : 2 : 5	1 : 2 : 5
Weight, in pounds per cubic foot.	113	113	.....	.....	.....
One-month test:					
Crushing strength, in pounds per square inch	1 700	1 366	1 389	1 474	1 142
Modulus of elasticity, in pounds per square inch	1 884 000	1 486 000	1 156 000	1 345 000	1 430 000
Two-month test:					
Crushing strength, in pounds per square inch	2 027	2 104	.....	.....	.....
Modulus of elasticity, in pounds per square inch	1 832 000	1 774 000	.....	.....	.....
One-year test:					
Crushing strength, in pounds per square inch	2 570	.....	.....	.....	.....
Modulus of elasticity, in pounds per square inch	1 475 000	.....	.....	.....	.....

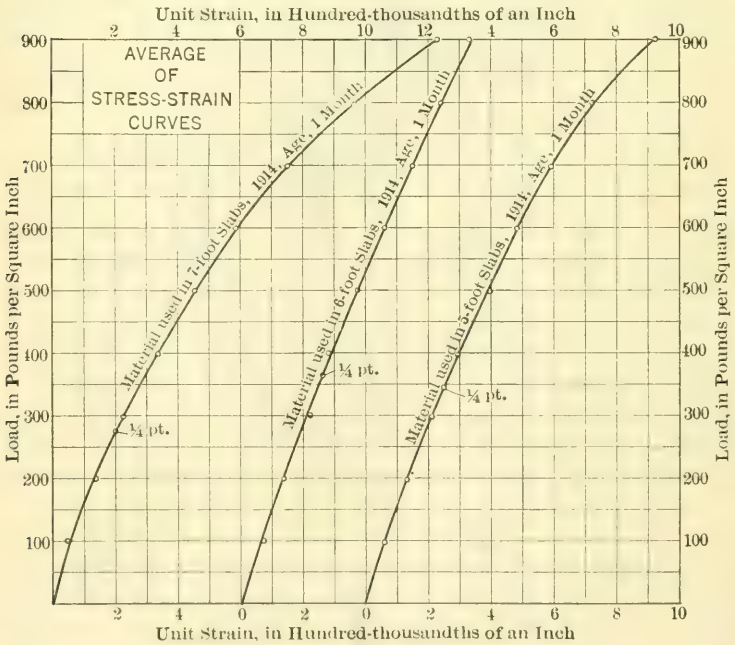

Unit Strain, in Hundred-thousandths of an Inch
AVERAGE OF STRESS-STRAIN CURVES
Material used in 7-foot Slabs, 1914, Age, 1 Month
Material used in 6-foot Slabs, 1914, Age, 1 Month
Material used in 5-foot Slabs, 1914, Age, 1 Month
¼ pt.
¼ pt.
¼ pt.
Load, in Pounds per Square Inch
Load, in Pounds per Square Inch
Unit Strain, in Hundred-thousandths of an Inch

Fig. 31.

APPENDIX III

Typical Slabs.

Full Log Sheets and Curves.

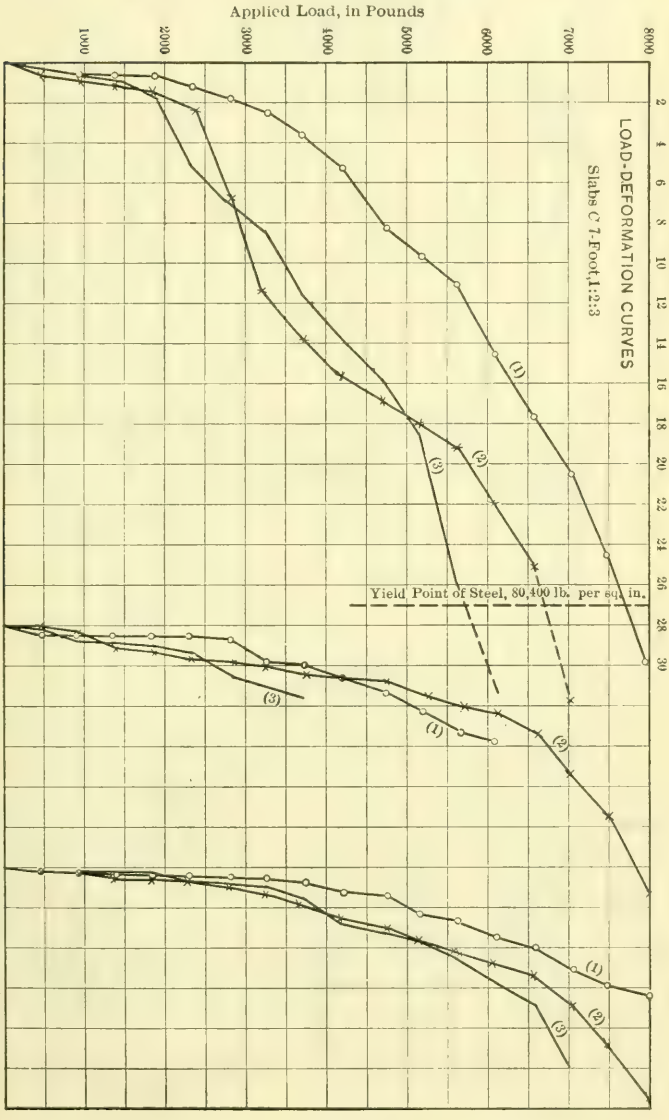

Applied Load, in Pounds
1000
2000
3000
4000
5000
6000
7000
8000
LOAD-DEFORMATION CURVES
Slabs C 7-Foot, 1:2:3
Yield Point of Steel, 80,400 lb. per sq. in.
(1)
(2)
(3)
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6 8 10 12 14 16 18 20
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4 6 8 10
Deflection, in Inches
0.2 0.4 0.6 0.8 1.0

Fig. 32.

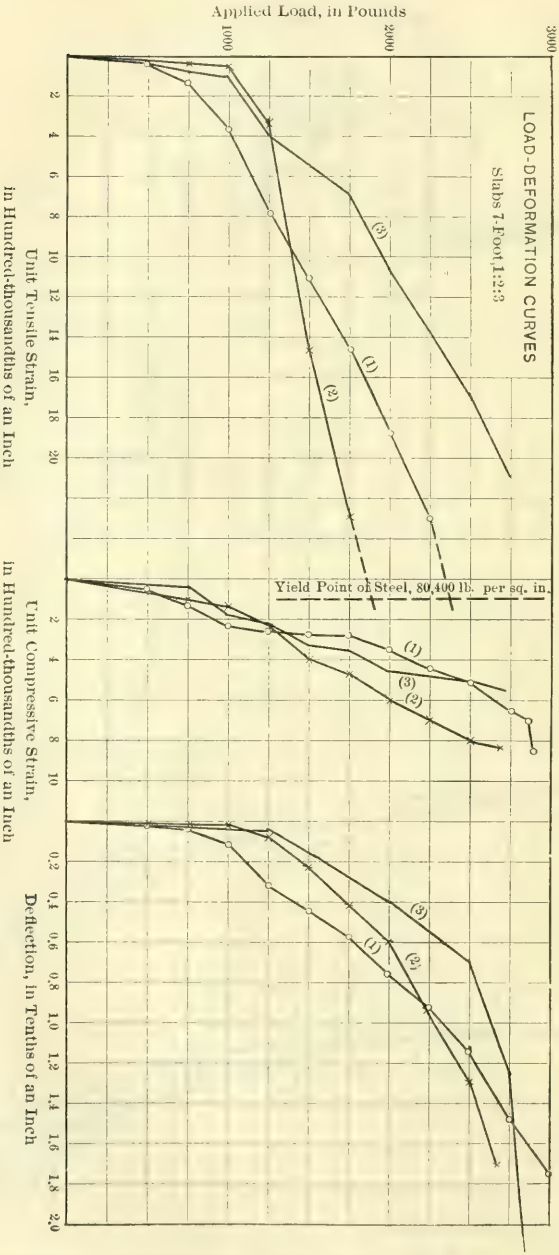

LOAD-DEFORMATION CURVES
Slabs 7-Foot 1:2:3
Applied Load, in Pounds
1000
2000
3000
Yield Point of Steel, 80,400 lb. per sq. in.
(1)
(2)
(3)
Unit Tensile Strain, in Hundred-thousandths of an Inch
Unit Compressive Strain, in Hundred-thousandths of an Inch
Deflection, in Tenths of an Inch

FIG. 33.

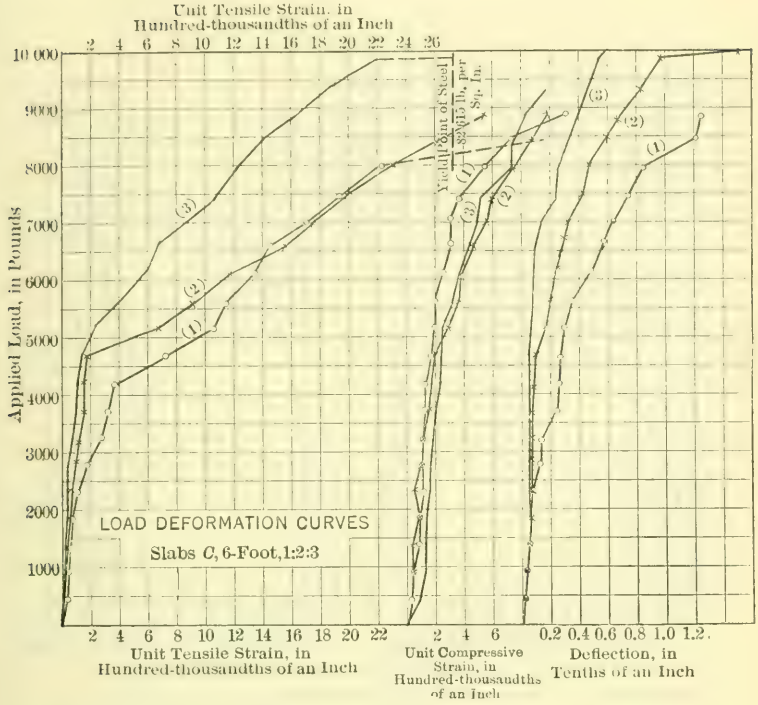

Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6 8 10 12 14 16 18 20 22 24 26
10 000
9000
8000
7000
6000
5000
4000
3000
2000
1000
Applied Load, in Pounds
Yield Point of Steel 82615 lb. per Sq. In.
(1)
(2)
(3)
LOAD DEFORMATION CURVES
Slabs C, 6-Foot, 1:2:3
2 4 6 8 10 12 14 16 18 20 22
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6
Unit Compressive Strain, in Hundred-thousandths of an Inch
0.2 0.4 0.6 0.8 1.0 1.2
Deflection, in Tenths of an Inch

FIG. 34.

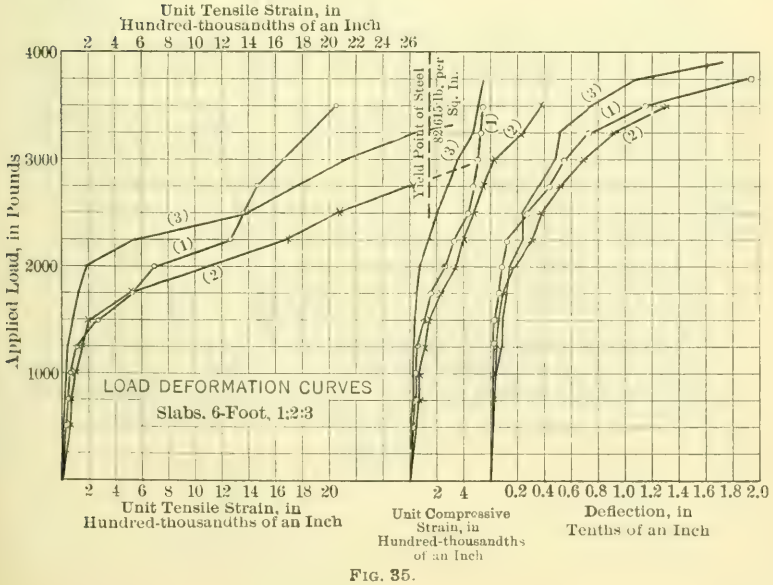

Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6 8 10 12 14 16 18 20 22 24 26
4000
3000
2000
1000
Applied Load, in Pounds
Yield Point of Steel 82615 lb. per Sq. In.
(1)
(2)
(3)
LOAD DEFORMATION CURVES
Slabs. 6-Foot, 1:2:3
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4
Unit Compressive Strain, in Hundred-thousandths of an Inch
0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0
Deflection, in Tenths of an Inch

FIG. 35.

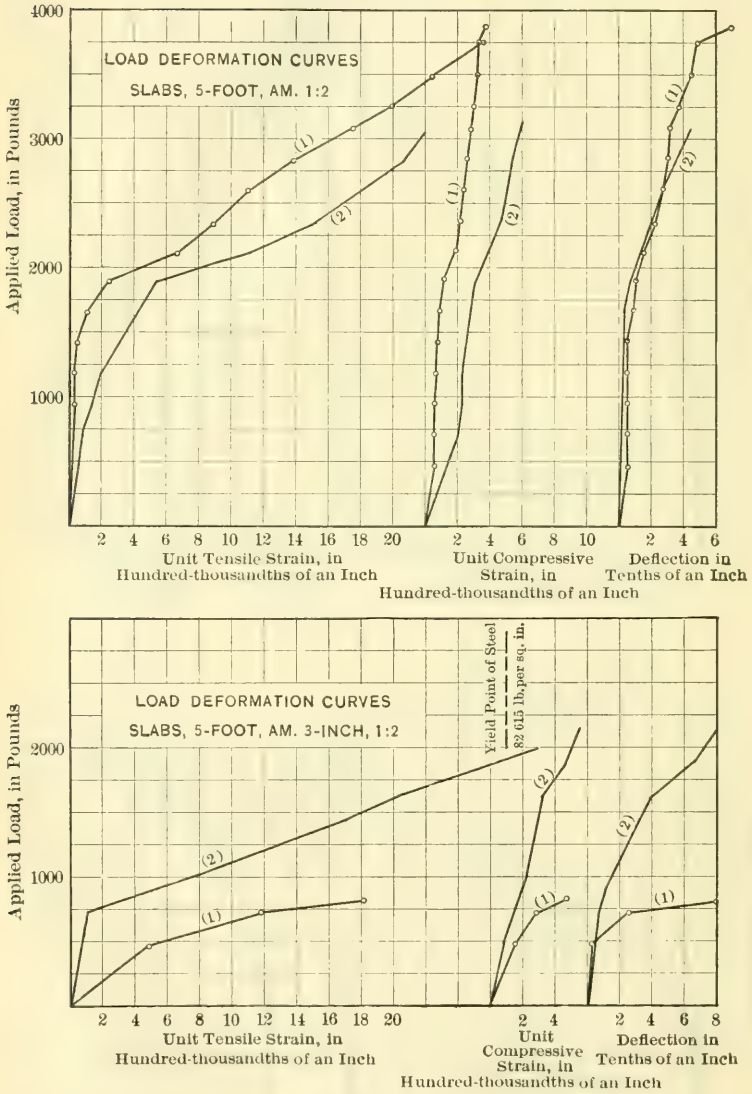

LOAD DEFORMATION CURVES
SLABS, 5-FOOT, AM. 1:2
Applied Load, in Pounds
4000
3000
2000
1000
(1)
(2)
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6 8 10
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4 6
Deflection in Tenths of an Inch
LOAD DEFORMATION CURVES
SLABS, 5-FOOT, AM. 3-INCH, 1:2
Yield Point of Steel
82 615 lb. per sq. in.
Applied Load, in Pounds
2000
1000
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4 6 8
Deflection in Tenths of an Inch

FIG. 36.

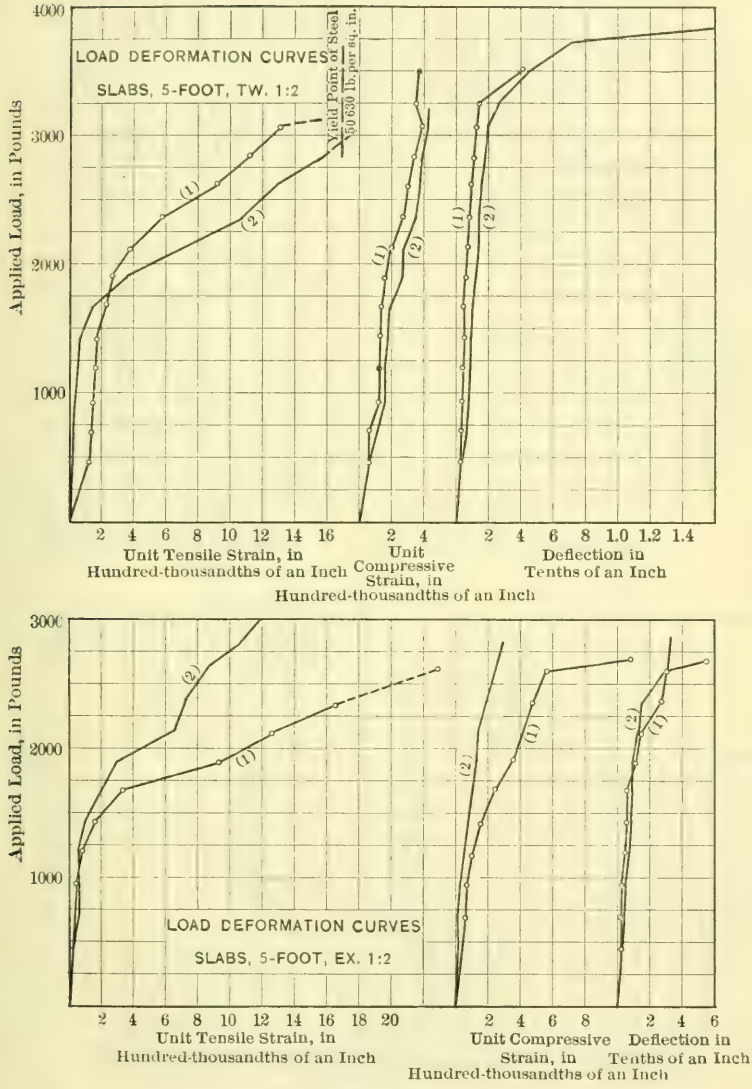

LOAD DEFORMATION CURVES
SLABS, 5-FOOT, TW. 1:2
Yield Point of Steel
50,630 lb. per sq. in.
Applied Load, in Pounds
4000
3000
2000
1000
(1)
(2)
2 4 6 8 10 12 14 16
Unit Tensile Strain, in
Hundred-thousandths of an Inch
2 4
Unit
Compressive
Strain, in
Hundred-thousandths of an Inch
2 4 6 8 1.0 1.2 1.4
Deflection in
Tenths of an Inch
LOAD DEFORMATION CURVES
SLABS, 5-FOOT, EX. 1:2
3000
2000
1000
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in
Hundred-thousandths of an Inch
2 4 6 8
Unit Compressive
Strain, in
Hundred-thousandths of an Inch
2 4 6
Deflection in
Tenths of an Inch

FIG. 37.

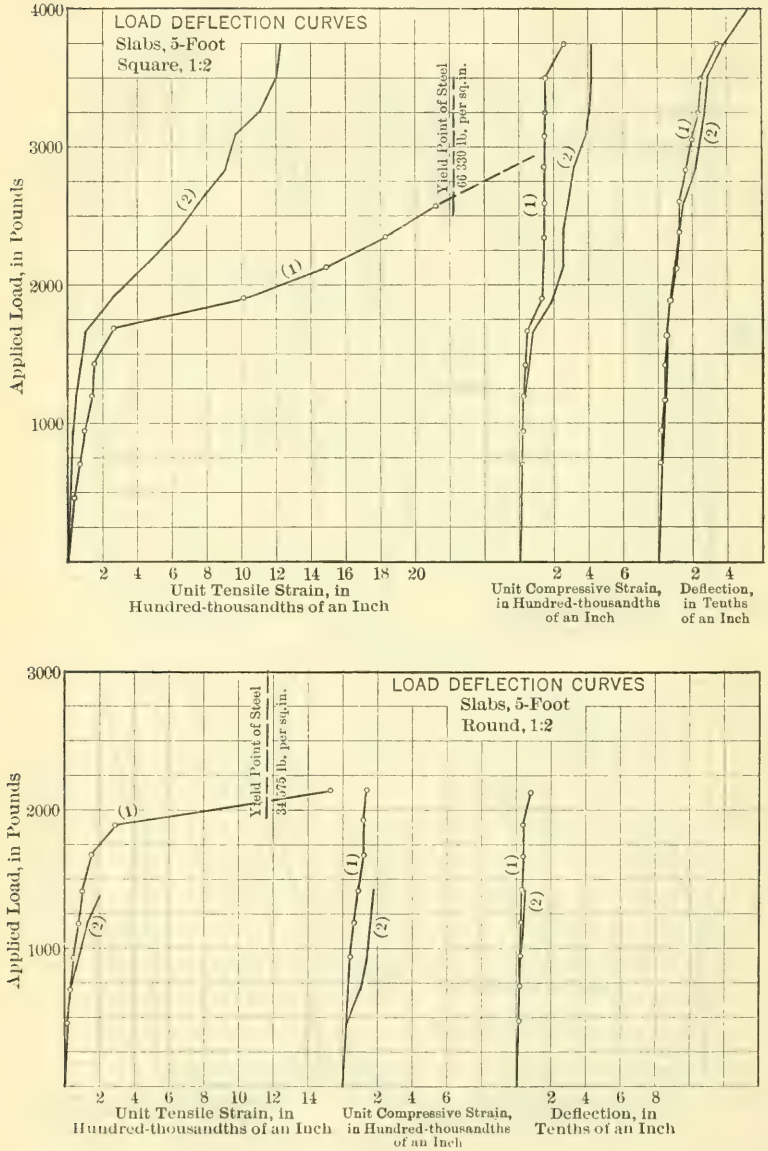

LOAD DEFLECTION CURVES
Slabs, 5-Foot
Square, 1:2
Applied Load, in Pounds
4000
3000
2000
1000
Yield Point of Steel
66 330 lb. per sq. in.
(1)
(2)
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in
Hundred-thousandths of an Inch
2 4 6
Unit Compressive Strain,
in Hundred-thousandths
of an Inch
2 4
Deflection,
in Tenths
of an Inch
LOAD DEFLECTION CURVES
Slabs, 5-Foot
Round, 1:2
Applied Load, in Pounds
3000
2000
1000
Yield Point of Steel
34 575 lb. per sq. in.
2 4 6 8 10 12 14
Unit Tensile Strain, in
Hundred-thousandths of an Inch
2 4 6
Unit Compressive Strain,
in Hundred-thousandths
of an Inch
2 4 6 8
Deflection, in
Tenths of an Inch

FIG. 38.

TABLE 13.—Log of Slab Tests.

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
C–7-ft., 1 : 2 : 3–R.												
475	0.02	0.03	0.02	.....	7	.....	4	.....	2			
950	0.03	0.03	0.02	5	10	5	4	3	6			
1 425	0.04	0.05	0.02	6	13	9	4	10	.....			
1 900	0.04	0.06	0.03	6	16	19	4	13	10			
2 350	0.04	0.07	0.07	11	22	50	4	17	15			Crack over haunch A-test span.
2 825	0.05	0.09	0.08	19	66	72	6	19	24			Crack 2 in. inside roller B and 4 in. from mid-tension side.
3 250	0.06	0.13	0.10	25	114	82	18	20	29		(Crack over haunch B-test span.)	
3 725	0.08	0.19	0.16	37	139	115	20	23	35			Crack over haunch B.
4 175	0.12	0.25	0.27	54	156	136	26	26	148	Crack over haunch A-test span.	Crack mid-tension side.	Central crack extends across.
4 675	0.15	0.31	0.32	81	167	160	32	29	151	Crack over haunch B-test span.		Crack 4 in. from roller A.
5 175	0.24	0.37	0.36	98	180	204	42	34	169	(Above cracks extending across top of slab.)	Crack under roller A.	Crack at roller extends across.
5 625	0.27	0.43	0.48	110	191	260	52	40	184	Cracks mid-tension side.	(Crack 5 in. from mid-tension side extending across underside test span.)	Crack 5 in. from mid.
6 100	0.34	0.48	0.58	146	219	310	58	42	204	Cracks under rollers.		Crack mid. top anchor-span A.
6 575	0.40	0.55	0.68	178	251	374	148	52	234		First crack $\frac{1}{16}$ in. wide.	Crack mid. top anchor-span B.
7 025	0.50	0.67	1.00	205	318	446	154	75	.....			
7 475	0.59	0.88	1.86	247	400	605	164	94	.....			
7 975	0.63	1.15	.....	299	600	.....	182	131	.....			
8 175	.....	1.70	.....	354	.....	.....	.....	156	.....			
8 450	0.88	.....	.....	480	.....	.....	202	.....	.....			
8 700	1.22	.....	.....	640	.....	.....	248	.....	.....	After passing maximum slab carried less load wire fracturing at 7 475 lb. Concrete then crushed over central crack.	Crushing of concrete at crack 5 in. from mid-test span.	Crushing of concrete at crack 5 in. from mid-test span.

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
C–6-ft. 1 : 2 : 3–R.												
475	0.01	0.02	0.02	4	1	1	2	.....	9			
950	0.03	0.04	0.04	5	2	1	4	3	11			
1 425	0.03	0.04	0.04	6	2	3	6	.....	11			
1 900	0.05	0.04	0.04	7	5	3	7	6	12			Crack at mid. tension side, test span.
2 350	0.06	0.06	0.04	11	8	3	12	6	14			
2 825	0.15	0.06	0.05	18	9	3	12	12	15			
3 250	0.16	0.06	0.05	25	10	6	13	13	17			
3 725	0.23	0.06	0.06	32	13	8	14	16	20		Crack at roller B—tension side.	
4 175	0.24	0.08	0.06	38	.....	8	14	.....	21	Crack extending across at middle.		Crack under roller B, tension side.
4 675	0.25	0.10	0.06	72	15	12	18	19	21		Crack over haunch test span.	
5 175	0.30	0.18	0.08	107	66	22	20	29	23		Crack 2 in. from mid. ten. side.	Cracks over both haunches, test span
5 625	0.35	0.20	0.10	116	94	42	20	36	31	Crack under roller A.		Crack 7 in. from mid. tension side, test span.
6 100	0.47	0.22	0.11	136	116	60	25	37	36	Crack over both haunches on test span and top of anchor-spans.		Crack under roller A—tension side.
6 575	0.55	0.28	0.12	146	156	69	30	45	43		Crack in anchor-span A.	
7 025	0.62	0.32	0.15	171	176	89	30	55	49		Crack in anchor-span B.	
7 475	0.73	0.41	0.22	194	198	107	37	59	52		Crack 2 in. from center.	
7 975	0.85	0.46	0.24	224	230	122	55	75	69	Crack across tension side, 5 in. from middle.		
8 450	1.21	0.60	0.32	331	262	140	72	88	69			
8 675	1.25	.....	.....	416	.....	.....	112	.....	.....			
8 875	.....	0.68	0.40	.....	295	161	.....	99	73			
9 375	.....	0.84	0.46	.....	.....	.....	.....	122	97			
9 920	.....	1.08	0.55	.....	.....	170	.....	171	.....			
100 075	.....	1.55	.....	.....	.....	.....	.....	.....	.....			
10 425	.....	.....	0.62	.....	.....	243	.....	.....	98			
10 920	.....	.....	0.89	.....	.....	262	.....	.....	109			

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
7–FT., 1 : 2 : 3–F.												
500	0.02	0.01	.....	3	0	3	6	0	3			
750	0.04	0.01	.....	12	2	7	13	12	.....			
1 000	0.12	0.01	.....	37	4	10	23	14	18			
1 250	0.32	0.09	0.05	79	35	40	28	26	24	Crack mid-tension side.	Crack mid-tension side.	Crack 2 in. outside roller A.
1 500	0.43	0.21	0.15	111	143	55	28	40	34	Crack extends across.	Crack extends across.	Central crack extending across.
1 750	0.58	0.41	0.27	146	226	70	29	48	36		Crack of one-third point.	
2 000	0.77	0.59	0.40	186	281	108	37	60	45		Crack of one quarter point.	Crack 3 in. inside roller B.
2 250	0.92	0.93	0.56	228	344	138	44	70	48	Crack of one third points.		Crack 5 in. from center.
2 500	1.15	1.28	0.70	302	402	170	51	80	52			Crack 7 in. from center.
2 650	.....	1.63	.....	.....	.....	.....	.....	82	.....			Crack under roller A.
2 750	1.48	.....	1.25	339	.....	210	65	.....	55			
2 870	1.78	.....	2.15	.....	.....	.....	70	.....	.....			
2 900	.....	.....	.....	.....	.....	.....	84	.....	.....			
										Crushed under roller.	Tension failure.	Tension failure.

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.	
	1	2	3	1	2	3	1	2	3	1	2
6-ft. 1 year.–1 : 2.											
500	0.00	.....	0.01	0	.....	1	0	.....	0		
750	0.01	.....	0.01	0	.....	2	2	.....	1		
1 000	0.01	.....	0.03	0	.....	3	2	.....	2		
1 250	0.01	.....	0.03	0	.....	4	2	.....	5		
1 500	0.02	.....	0.05	0	.....	5	8	.....	5		
1 750	0.02	.....	0.06	1	.....	6	9	.....	6		
2 000	0.02	.....	0.08	2	.....	8	.....	.....	12		
2 250	0.12	.....	0.26	64	.....	15	14	.....	12	Crack under roller B	Cracks 3 in. inside each roller.
2 500	0.27	.....	0.42	119	.....	163	26	.....	24	Crack 1½ in. from mid.	
2 750	0.40	.....	0.57	230	.....	215	40	.....	39		
3 000	0.55	.....	0.64	283	.....	256	49	.....	41	Crack under roller A.	
3 250	0.62	.....	0.74	320	.....	306	52	.....	44	Crack 3 in. from mid.	
3 500	0.75	.....	0.89	424	.....	406	54	.....	52		Cracks 5 in. outside roller A.
3 750	0.95	.....	1.31	482	.....	.....	56	.....	70		Cracks 4 in outside roller B.
3 800	.....	.....	1.61	.....	.....	.....	.....	.....	.....		
4 000	1.50	.....	.....	844	.....	.....	61	.....	.....		
4 138	.....	.....	.....	.....	.....	.....	.....	.....	.....		

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
6-ft. 1 : 2 : 3.												
500	0.00	0.00	0.00	2	4	1	2	0	1			
750	0.00	0.01	0.01	3	6	2	4	5	3			
1 000	0.00	0.02	0.03	5	8	3	4	6	3			
1 250	0.01	0.02	0.03	10	.....	4	7	14	4		Crack mid. tension side.	
1 500	0.01	0.03	0.05	25	21	6	11	16	6			
1 750	0.01	0.07	0.06	54	53	11	14	22	7		Crack 7 in. from middle.	
2 000	0.01	0.15	0.10	70	.....	28	29	32	13	Crack tension side, mid.	Crack under roller B.	
2 250	0.01	0.30	0.12	128	166	56	32	39	19	Crack tension side, under both rollers.		Crack 10 in. from mid. and at mid.
2 500	0.26	0.37	0.22	137	201	140	41	46	23			Crack 8 in. from mid.
2 750	0.42	0.51	0.23	146	263	171	48	52	28	Crack 7 in. from middle.		Crack under roller A.
3 000	0.57	0.69	0.38	168	389	211	51	62	36	Crack 7 in. from middle, opposite end.	Crack outside roller A.	
3 250	0.74	0.91	0.50	.....	453	262	53	81	47	Crack 7 in. outside roller A.		
3 500	1.18	1.31	0.52	203	654	318	56	95	51	" 8 in. " " B.		
3 550	.....	.....	0.74	.....	.....	.....	.....	.....	.....			
3 750	1.97	.....	1.07	.....	.....	398	.....	.....	55	Fractured wire.		
3 900	.....	.....	1.72	.....	.....	.....	.....	.....	.....			

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain, in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks.	
	1	2	1	2	1	2	1	2
5–FT. EX.–1 : 2.								
475	0.02	0.02	.....	1	.....	1		
725	0.02	0.03	.....	4	6	1		
950	0.02	0.04	3	4	7	2		
1 200	0.04	0.06	7	5	10	5		
1 425	0.05	0.07	16	10	11	7		
1 675	0.05	0.09	33	19	23	10		
1 900	0.11	0.10	94	28	35	15		
2 125	0.15	0.13	126	64	.....	16	Crack at mid. tension side.	
2 350	0.24	0.14	165	74	47	19		
2 600	0.30	0.30	228	87	55	23		Crack 3 in. from mid.
2 690	0.55	.....	.....	.....	107	.....		
2 825	.....	0.33	.....	108	.....	27		
3 005	.....	.....	.....	118	.....	23		

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks.	
	1	2	1	2	1	2	1	2
5-FT. AM. 1 : 2.								
475	0.04	0.01	.....	6	5	14		Tension reading taken on a 12-in. gauge length; compression on a 10-in. gauge length.
725	0.04	0.02	.....	8	5	20		
950	0.05	0.02	1	14	5	23		
1 200	0.05	0.03	2	19	7	23		
1 425	0.06	0.04	3	30	8	25		
1 675	0.08	0.05	10	42	9	28		
1 900	0.10	0.08	24	72	12	31		
2 125	0.15	0.14	65	110	19	40	Crack under roller B, tension side.	Crack under roller A, tension side.
2 350	0.22	0.20	89	149	21	47		Crack at mid., tension side.
2 600	0.26	0.26	110	179	23	52		Crack 3 in. outside roller B, tension side.
2 825	0.29	0.35	138	206	26	55		
3 050	0.32	0.44	176	220	29	60		
3 250	0.37	.....	199	.....	31	.....	Crack 4 in. outside roller A, also at mid.	
3 500	0.44	.....	225	.....	34	.....	Crack 8 in. from mid.	
3 725	0.48	.....	255	.....	34	.....		
3 875	0.72	.....	298	.....	39	.....		
5-FT. 3-IN. AM. 1 : 2.								
475	0.02	0.05	48	6	16	8		
725	0.24	0.07	118	10	28	14	Crack 2 in. from mid., also 2 in. outside roller A.	
81	0.82	.....	181	.....	47	.....	Crack 1 in. outside roller A.	
955	.....	0.13	.....	65	.....	26	Crack under roller B.	Crack under both rollers, tension side.
1 200	.....	0.22	.....	116	.....	.....		Crack at middle.
1 420	.....	0.30	.....	160	.....	28		
1 675	.....	0.40	.....	201	.....	34		
1 905	.....	0.66	.....	260	.....	46		
2 120	.....	0.81	.....	362	.....	56		
2 215	.....	.....	.....	410	.....	56		

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain, in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks.	
	1	2	1	2	1	2	1	2
5-ft. Rd.—1 : 2 : 3.								
475	0.01	.....	1	1	0	2		
725	0.01	.....	2	2	0	6		
950	0.02	0.01	4	5	3	9		
1 200	0.02	0.03	6	8	6	11		
1 425	0.03	0.04	9	20	9	13		Crack 7 in. from middle.
1 575	.....	.....	.....	.....	.....	.....		
1 675	0.03	.....	16	.....	12	.....		
1 900	0.04	.....	28	.....	12	.....		
2 125	0.06	.....	154	.....	14	.....	Crack 2 in. from middle. Bond failure.	Bond failure.
7-ft. Am. Special.								
475	.....	.....	.....	.....	.....	.....		
950	.....	.....	.....	.....	.....	.....		
1 425	.....	.....	.....	.....	.....	.....		
1 900	.....	0.01	.....	.....	.....	.....		
2 125	.....	.....	.....	.....	.....	.....	Crackling sounds at free end.	
2 350	.....	0.04	.....	.....	.....	.....		
2 825	.....	0.08	.....	.....	.....	.....		
3 250	.....	0.12	.....	.....	.....	.....	Crack across fixed end test slab 4 in. from beam.	
3 725	.....	0.22	.....	.....	.....	.....	Cracks under roller at free end and at free haunch fillet. Crack 6 in. toward free end from mid. Crack between mid. and roller at free end.	
4 175	.....	0.56	.....	.....	.....	.....	Crack under roller at fixed end. Free end haunch crack open 1/16 in. Overhang at top of beam at free end lifting.	
4 675	.....	0.63	.....	.....	.....	.....	Crushing of concrete under roller at free end. Wires fractured in test span and crack appeared 1 ft. from beam on top of unloaded span.	

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain, in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks	
	1	2	1	2	1	2	1	2
5-ft. Tw. 1 : 2								
475	0.02	0.02	14	2	6	6		
725	0.02	0.03	16	2	6	11		
920	0.02	0.04	16	4	12	15		
1 200	0.03	0.05	17	6	12	15		
1 425	0.04	0.05	18	9	12	18		
1 675	0.04	0.07	22	15	13	19		
1 900	0.04	0.10	26	35	16	25		
2 125	0.06	0.12	38	74	20	26		
2 350	0.06	0.12	58	110	26	33		
2 600	0.08	0.14	92	131	30	37		
2 825	0.10	0.17	112	158	33	38		Cracks at middle and under roller A.
3 050	0.12	0.20	130	187	39	41	Cracks 2 in. outside roller A.	
3 250	0.15	0.25	190	287	36	41	Cracks 2 in. inside roller B.	
3 500	0.42	0.43	190	529	37	24	Cracks 3 in. inside roller A.	
3 545	.....	0.74	.....	.....	.....	8		Crack 5 in. from center.
3 590	.....	1.62	.....	.....	.....	.....		
5-ft. Sq. 1 : 2								
475	.....	.....	3	1	0	.....		
725	0.01	0.01	5	2	1	.....		
950	0.01	0.02	8	2	1	.....		
1 200	0.02	0.02	13	4	2	.....		
1 425	0.02	0.03	16	6	4	5		
1 675	0.03	0.04	26	9	6	9		
1 900	0.05	0.04	101	24	13	18		
2 125	0.10	0.09	148	42	.....	23		
2 350	0.13	0.12	182	62	15	23		
2 600	0.16	0.16	212	76	15	27		
2 825	0.18	0.22	236	90	15	32		
3 050	0.20	0.24	266	98	15	39		
3 250	0.25	0.27	.....	110	15	34	Crack at middle.	Crack 1 in. outside roller B and at middle.
3 500	0.28	0.32	.....	120	.....	41	Crack 5 in. outside roller A and 2 in. outside roller B.	Crack at roller A.
3 725	0.33	0.38	.....	124	25	41		
3 825	.....	0.48	.....	.....	.....	37		Crack 7 in outside roller B.

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Deflection, in Inches.							
	1	2						
5–FT. PLAIN.								
100	.....	.....	.....	.....	.....	.....		
200	.....	.....	.....	.....	.....	.....		
300	.....	.....	.....	.....	.....	.....		
400	.....	.....	.....	.....	.....	.....		
500	.....	.....	.....	.....	.....	.....		
600	.....	.....	.....	.....	.....	.....		
700	0.001	0.001	.....	.....	.....	.....		
800	0.0015	0.001	.....	.....	.....	.....		
900	0.002	0.0035	.....	.....	.....	.....		
1 000	.....	0.004	.....	.....	.....	.....		
1 100	0.010	0.0085	.....	.....	.....	.....		
1 200	0.012	0.0115	.....	.....	.....	.....		
1 300	0.015	0.014	.....	.....	.....	.....		
1 400	0.019	0.0175	.....	.....	.....	.....		
1 500	.....	0.0275	.....	.....	.....	.....		

APPENDIX IV

Tests of Various Mixtures of Cinder Concrete Cast and Tested Under Identical Conditions.

Method.—Machine-mixed concrete of medium consistency was cast in the form of standard cylinders, in sets of four. They were placed in water after 48 hours, removed from water 48 hours before testing, and faced with plaster of Paris; age, 28 days.

Materials.—

Dragon Portland cement.

Long Island sand.

Anthracite clinker, Columbia University power plant.

Results.—

TABLE 14.—Ultimate Compressive Strength, in Pounds Per Square Inch, at 28 Days.

Mixture.	Cinders as received.	Cinders finer than ¼-in. screened out.	Weight, in Pounds per Cubic Foot.	
			Unscreened.	Screened.
1 : 2 : 4	975	779	114	113
1 : 2 : 5	787	585	115	110
1 : 3 : 6	708	670	113	112
1 : 7	382	...	83	...

Remarks.—The results in Table 14, inclusive of weights, must be considered only as comparative, because, not only were the specimens stored in water, but the consistency also was rather dry.

APPENDIX V

DERIVATIONS OF EXPRESSIONS FOR BENDING MOMENTS AND SHEARS; AND MISCELLANEOUS.

Uniform Load.—Three Spans, Fig. 39.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) + M_3 l_2 = -6 \left(\frac{s_1 y_1}{1} + \frac{s_2 y_2}{2} \right) \quad \ldots\ldots(1)$$

where s_1 and s_2 are bending moment areas for simple span, and y_1 and y_2 are distances to supports from centers of gravity.

$$2 M_2 \left(\frac{l}{2} + l \right) + M_3 l = -\frac{1}{32} pl^3 - \frac{1}{4} pl^3$$

as $M_1 = M_4 = 0$ (free ends).

$$\text{or } 3 M_2 l + M_3 l = -\frac{9}{32} pl^3 \quad \ldots\ldots(2)$$

$$\text{Similarly, } M_2 l + 3 M_3 l = -\frac{9}{32} pl^3 \quad \ldots\ldots(3)$$

Solving Equations (2) and (3),

$$M_2 = M_3 = -\frac{9}{128} pl^2 = -\frac{1}{14.2} pl^2 \quad \ldots\ldots(4)$$

Taking moments of Span 1 about Support 2,

$$R_1 \times \frac{1}{2} l - \frac{1}{2} pl \times \frac{1}{4} l = -\frac{9}{128} pl^2.$$

Whence $$R_1 = +\frac{7}{64} pl \quad \ldots\ldots(5)$$

Taking moments of Span 1 about Support 1,

$$R_2 \times \frac{1}{2} l - \frac{9}{128} pl^2 - \frac{1}{8} pl^2 = 0.$$

Whence $$R_2' = +\frac{25}{64} pl \quad \ldots\ldots(6)$$

also $$R_2'' = +\frac{1}{2} pl \quad \ldots\ldots(7)$$

Taking moments about the center of Span 2,

$$M_c = \frac{1}{2} pl \times \frac{1}{2} l - \frac{9}{128} pl^2 - \frac{1}{8} pl^2$$

$$M_c = +\frac{7}{128} pl^2 = +\frac{1}{18.3} pl^2 \quad \ldots\ldots(8)$$

Taking moments about the one-third point of Span 2,

$$M_{\frac{1}{3}} = \frac{1}{2} pl \times \frac{1}{3} l - \frac{9}{128} pl^2 - \frac{1}{3} pl \times \frac{1}{6} l$$

$$M_{\frac{1}{3}} = +\frac{47}{1\,152} pl^2 = +\frac{1}{24.5} pl^2 \quad \ldots\ldots(9)$$

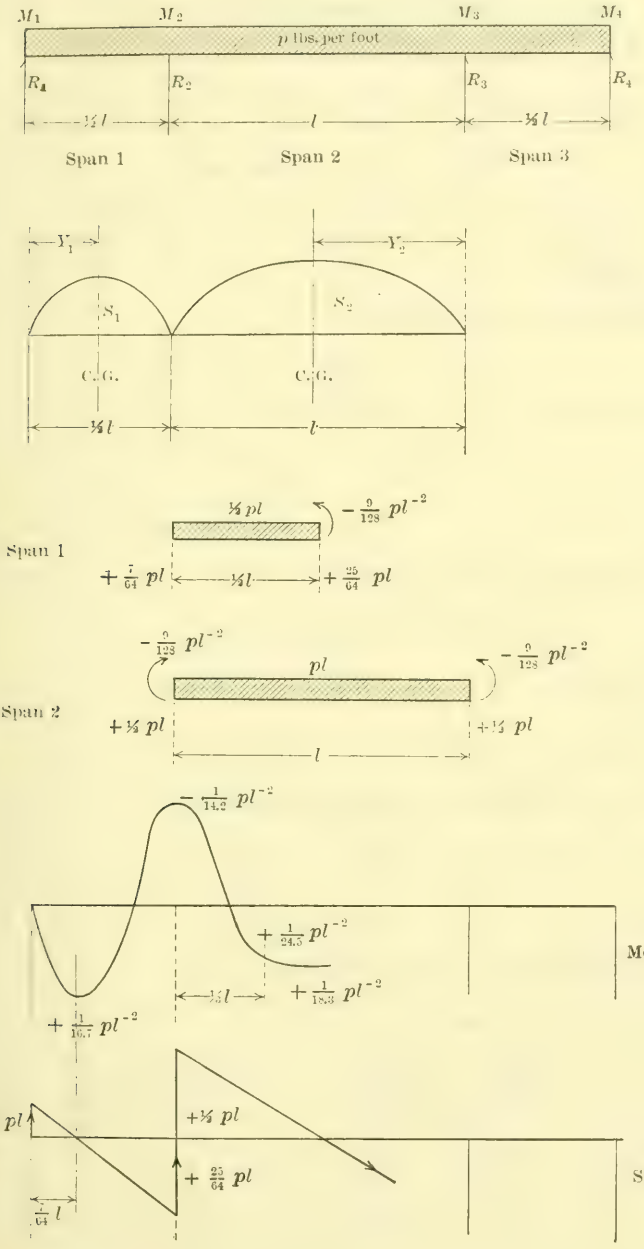

M_1
M_2
M_3
M_4
p lbs. per foot
R_1
R_2
R_3
R_4
.4 l
l
.4 l
Span 1
Span 2
Span 3
Y_1
Y_2
S_1
S_2
C.G.
C.G.
.4 l
l
.4 pl
$-\frac{9}{128}\ pl^{-2}$
Span 1
$+\frac{7}{64}\ pl$
.4 l
$+\frac{25}{64}\ pl$
$-\frac{9}{128}\ pl^{-2}$
pl
$-\frac{9}{128}\ pl^{-2}$
Span 2
+.4 pl
+.4 pl
l
$-\frac{1}{14.2}\ pl^{-2}$
$+\frac{1}{24.5}\ pl^{-2}$
Moments
.4 l
$+\frac{1}{18.3}\ pl^{-2}$
$+\frac{1}{16.7}\ pl^{-2}$
$+\frac{7}{64}\ pl$
+.4 pl
$+\frac{25}{64}\ pl$
Shears
$\frac{7}{64}\ l$

FIG. 39.

Third-Point Loads.—Three Spans, Fig. 40.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) + M_3 l_2 = -6\left(\frac{s_1 y_1}{1} + \frac{s_2 y_2}{2}\right) \quad \ldots\ldots(1)$$

Whence $2 M_2 \left(\frac{l}{2} + l\right) + M_3 l = -0 - \frac{1}{3} W l^2$

as $M_1 = M_4 = 0$ (free ends),

or $$3 M_2 l + M_3 l = -\frac{1}{3} W l^2 \quad \ldots\ldots(2)$$

Similarly, $$M_2 l + 3 M_3 l = -\frac{1}{3} W l^2 \quad \ldots\ldots(3)$$

Solving Equations (2) and (3),

$$M_2 = M_3 = -\frac{1}{12} W l \quad \ldots\ldots(4)$$

Taking moments of Span 1 about Support 2,

$$R_1 \times \frac{1}{2} = -\frac{1}{12} W l.$$

Whence $$R_1 = -\frac{1}{6} W \quad \ldots\ldots(5)$$

Taking moments of Span 1 about Support 1,

$$R_2' \times \frac{1}{2} l - \frac{1}{12} W l = 0.$$

Whence $$R_2' = +\frac{1}{6} W \quad \ldots\ldots(6)$$

Also $$R_2'' = +\frac{1}{2} W \quad \ldots\ldots(7)$$

Taking moments about the one-third point of Span 2,

$$M_{\frac{1}{3}} = \frac{1}{2} W \times \frac{1}{3} l - \frac{1}{12} W l$$

or $$M_{\frac{1}{3}} = +\frac{1}{12} W l \quad \ldots\ldots(8)$$

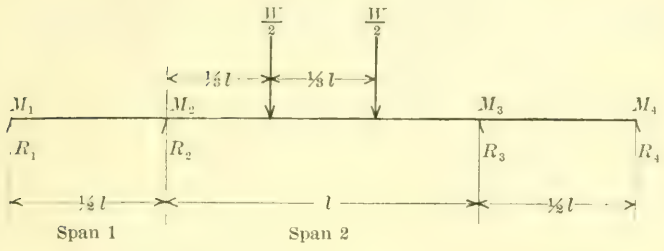
Uniform Load.—Free Span, Fig. 41.—

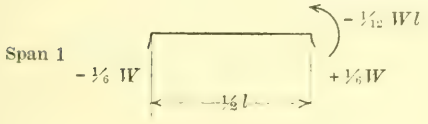
$$M_{center} = +\frac{1}{8} p l^2 \quad \ldots\ldots(1)$$

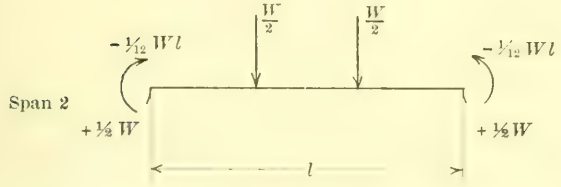
$$M_{\frac{1}{3}} = \frac{1}{2} p l \times \frac{1}{3} l - \frac{1}{3} p l \times \frac{1}{6} l$$

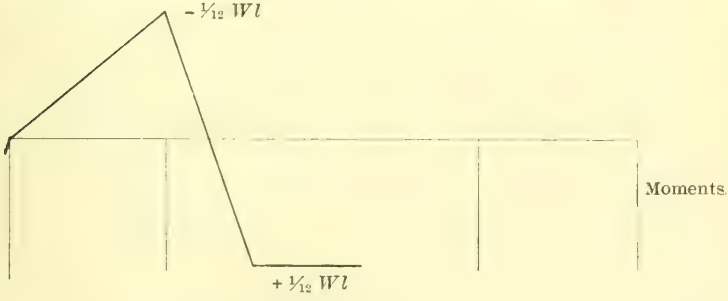
or $$M_{\frac{1}{3}} = +\frac{1}{9} p l^2 \quad \ldots\ldots(2)$$

$$R_1 = +\frac{1}{2} p l \quad \ldots\ldots(3)$$


W/2
W/2
⅓ l
⅓ l
M1
M2
M3
M4
R1
R2
R3
R4
½ l
l
½ l
Span 1
Span 2


- 1/12 Wl
Span 1
- 1/6 W
+ 1/6 W
½ l


W/2
W/2
- 1/12 Wl
- 1/12 Wl
Span 2
+ ½ W
+ ½ W
l


- 1/12 Wl
Moments
+ 1/12 Wl

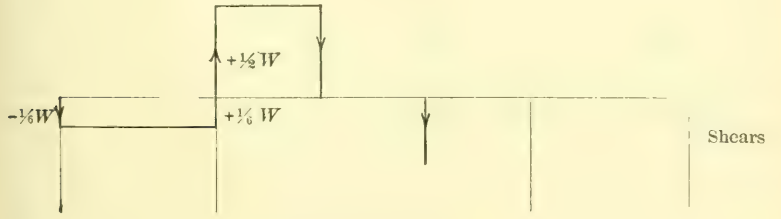

+½W
-⅙W
+⅙ W
Shears

FIG. 40.

Third-Point Loads.—Free Span, Fig. 42.—

$$M_{\frac{1}{3}} = M_{center} = + \frac{1}{6} W \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$R_1 = + \frac{1}{2} W \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

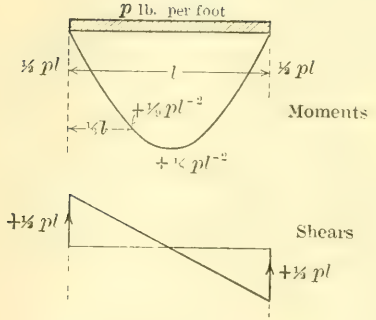


FIG. 41.

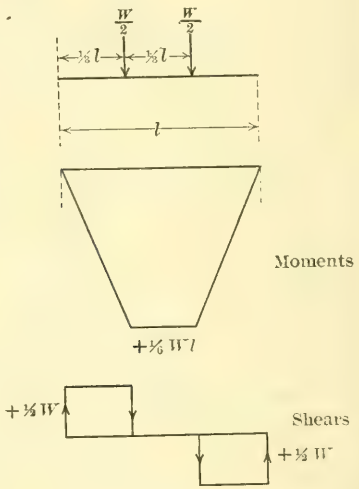


FIG. 42.

Uniform Load.—Two Spans, Special, Fig. 43.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) M_3 l_2 = - 6 \left(\frac{s_1 y_1}{l_1} + \frac{s_2 y_2}{l_2} \right) \ldots\ldots (1)$$

Whence $$2 M_2 (l + l) = - \frac{1}{4} pl^3 - \frac{1}{4} pl^3$$

As $M_1 = M_3 = 0$

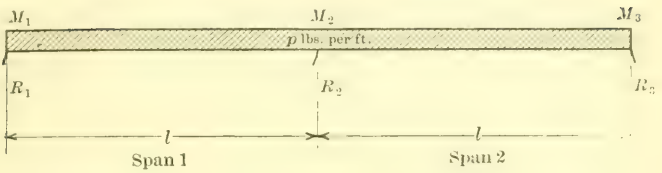
or $$4 M_2 l = - \frac{1}{2} pl^3 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

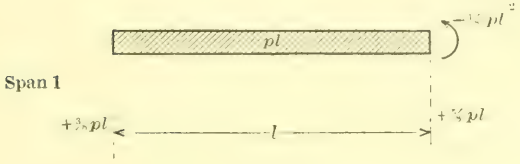
Whence $$M_2 = - \frac{1}{8} pl^2 \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

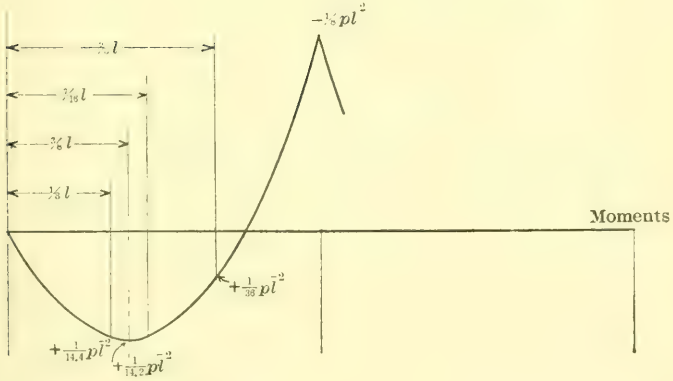
Taking moment of Span 1 about Support 2,

$$R_1 l - \frac{1}{2} pl^2 = - \frac{1}{8} pl^2$$

or $$R_1 = + \frac{3}{8} pl \ldots\ldots\ldots\ldots\ldots\ldots (4)$$


M_1
M_2
M_3
p lbs. per ft.
R_1
R_2
R_3
l
l
Span 1
Span 2


pl
Span 1
$+\frac{3}{8}pl$
l
$+\frac{5}{8}pl$


$-\frac{1}{8}pl^2$
$\frac{3}{8}l$
$\frac{7}{16}l$
$\frac{3}{8}l$
$\frac{1}{8}l$
Moments
$+\frac{1}{16}pl^2$
$+\frac{1}{14.4}pl^2$
$+\frac{1}{14.2}pl^2$

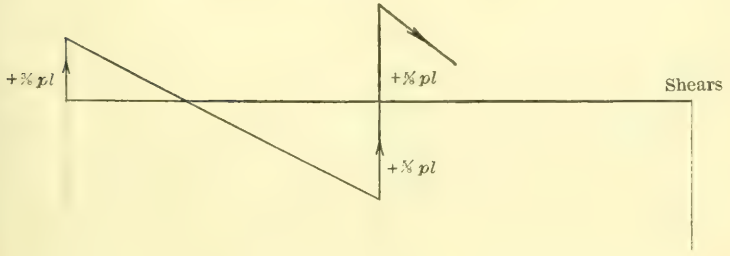

$+\frac{5}{8}pl$
$+\frac{5}{8}pl$
Shears
$+\frac{3}{8}pl$

FIG. 43.

Taking moment of Span 1 about Support 1,

$$R_2' \, l - \frac{1}{8} \, pl^2 - \frac{1}{2} \, pl^2 = 0$$

or $$R_2' = + \frac{5}{8} \, pl \quad \ldots\ldots (5)$$

and $$R_2'' = + \frac{5}{8} \, pl \quad \ldots\ldots (6)$$

Taking moments about the one-third point of Span 1,

$$M_{\frac{1}{3}} = \frac{1}{8} \, pl^2 - \frac{1}{18} \, pl^2$$

$$M_{\frac{1}{3}} = \frac{5}{72} \, pl^2 = + \frac{1}{14.4} \, pl^2 \quad \ldots\ldots (7)$$

Taking moments about the two-thirds point of Span 1,

$$M_{\frac{2}{3}} = \frac{3}{8} \, pl \times \frac{2}{3} \, l - \frac{2}{3} \, pl \times \frac{1}{3} \, l$$

$$M_{\frac{2}{3}} = \frac{1}{4} \, pl^2 - \frac{2}{9} \, pl^2 + \frac{1}{36} \, pl^2 \quad \ldots\ldots (8)$$

Taking moments about the three-eighths point of Span 1,

$$M_{\frac{3}{8}} = \frac{9}{64} \, pl^2 - \frac{9}{128} \, pl^2$$

$$M_{\frac{3}{8}} = \frac{9}{128} \, pl^2 = + \frac{1}{14.2} \, pl^2 \quad \ldots\ldots (9)$$

Third-Point Loads.—Two Spans, Special, Fig. 44.—

$$M_1 \, l_1 + 2 \, M_2 \, (l_1 + l_2) \; M_3 \, l_2 = - 6 \left(\frac{s_1 \, y_1}{1} + \frac{s_2 \, y_2}{2} \right) \quad \ldots\ldots (1)$$

Whence $$4 \, M_2 \, l = - \frac{1}{3} \, W \, l^2$$

as $$M_1 = M_3 = 0$$

and $$M_2 = - \frac{1}{12} \, W \, l \quad \ldots\ldots (2)$$

Taking moments of Span 1 about Support 2,

$$R_1 \times l - \frac{W}{2} \, l = - \frac{1}{12} \, W \, l$$

or $$R_1 = + \frac{5}{12} \, W \quad \ldots\ldots (3)$$

Taking moments of Span 1 about Support 1,

$$R_2' \times l - \frac{W}{2} \, l - \frac{1}{12} \, W \, l = 0$$

or $$R_2' = + \frac{7}{12} \, W$$

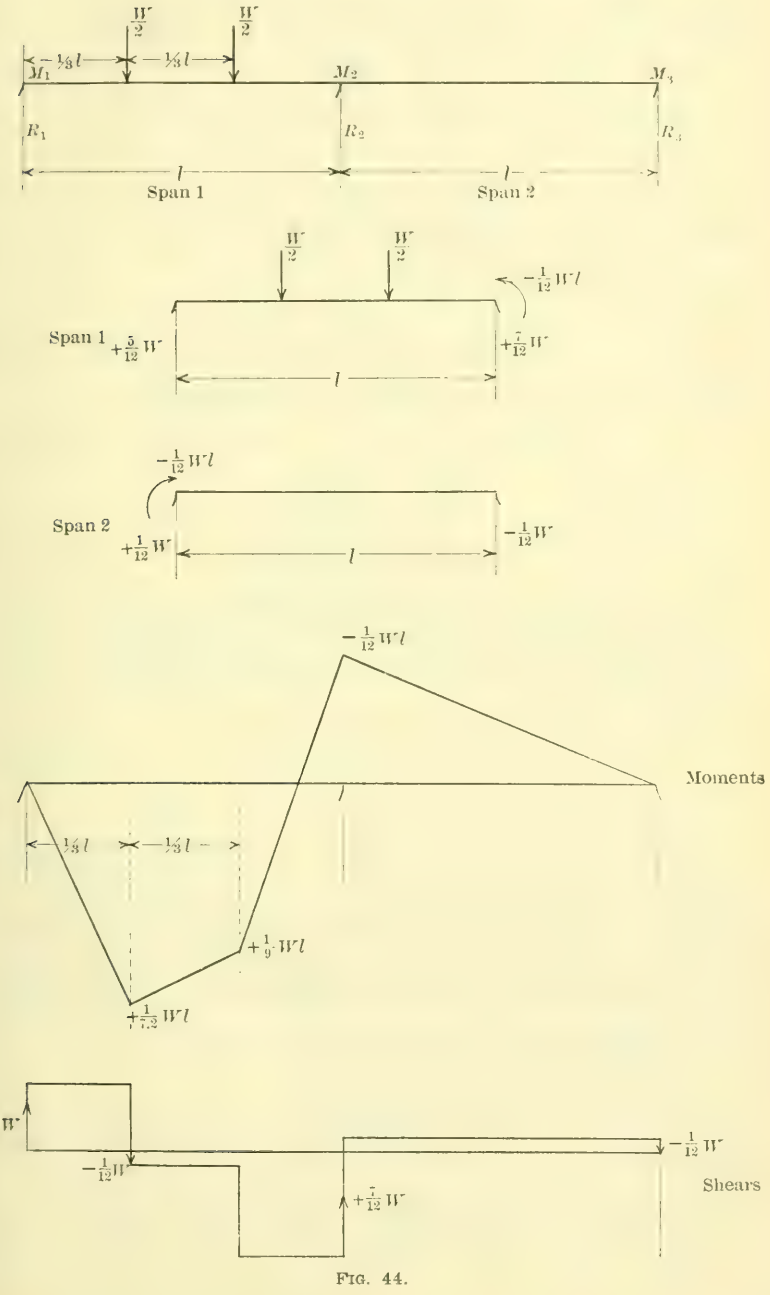

W/2
W/2
⅓ l
⅓ l
M1
M2
M3
R1
R2
R3
l
l
Span 1
Span 2
W/2
W/2
−1/12 Wl
Span 1
+5/12 W
+7/12 W
l
−1/12 Wl
Span 2
+1/12 W
−1/12 W
l
−1/12 Wl
Moments
⅓ l
⅓ l
+1/9 Wl
+1/12 Wl
+5/12 W
−1/12 W
−1/12 W
+7/12 W
Shears

FIG. 44.

Taking moments of Span 2 about Support 3,

$$R_2'' \times l - \frac{1}{12} W = 0$$

or $$R_2'' = + \frac{1}{12} W \ldots\ldots (4)$$

and $$R_3 = - \frac{1}{12} W \ldots\ldots (5)$$

Taking moments of Span 1 about the one-third point,

$$M_{\frac{1}{3}} = \frac{5}{12} W \times \frac{1}{3} l \frac{5}{36} W l$$

or $$M_{\frac{1}{3}} = \frac{1}{7.2} W l \ldots\ldots (6)$$

Taking moments of Span 1 about the two-thirds point,

$$M_{\frac{2}{3}} = \frac{5}{12} W \times \frac{2}{3} l - \frac{1}{2} W \times \frac{1}{3} l$$

or $$M_{\frac{2}{3}} = + \frac{1}{9} W l \ldots\ldots (7)$$

Deflection Formulas.

The deflection at any point of a homogeneous beam can be expressed by the common theory of flexure in terms of the distance from the tangent to the elastic curve at any other point, and is usually expressed as follows:

$$\frac{dy}{dx} = \int_a^b \frac{M}{E I} \times dx \ldots\ldots (1)$$

or $$\frac{d^2 y}{dx^2} = \frac{M}{E I} \ldots\ldots (2)$$

The analysis leading to Equation (2) gives a simple graphic method for estimating the elastic curve for a specific loading of a homogeneous beam, by loading the beam with the corresponding bending moment polygon. This deflection for any point from the tangent at any other point may also be expressed as the statical moment of the area under the $\frac{M}{E I}$ curve about the first point, or

$$y = K l^2 \left(\frac{M}{E I} \right) \ldots\ldots (3)$$

This equation has been used by Mr. G. A. Maney in the following form,

$$y = \frac{K_1}{K_2} l^2 \left(\frac{M}{E I} \right) \ldots\ldots (4)$$

In this equation, K_1 is the coefficient of the factor, $\frac{W l^3}{E I}$, in the expression for the maximum deflection of a homogeneous beam, and K_2 is the coefficient of Wl, in the expression for the maximum bending moment. The values for K_1 and K_2, with the type of loading used in this investigation, are here developed.

In a homogeneous beam of constant cross-section, $E I$ is a constant and $\frac{e}{c}$, the ratio of the unit deformation to the distance from the neutral axis to the extreme fiber, varies directly as the bending moment. Hence, in a reinforced concrete beam,

$$\frac{M}{E I} = \frac{e}{c} = \frac{e_c}{c_c} = \frac{e_s}{c_s} \quad \frac{e_c + e_s}{c_c + c_s} \quad \frac{e_c + e_s}{d}$$

or, from Equation (4)

$$y = \frac{K_1}{K_2} \frac{l^2}{d} (e_c + e_s) \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

or

$$y = \frac{K_1}{K_2} \frac{l^2}{d E_s} (n f_c + f_s) \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Derivation of K_1 and K_2 on Three-Span Test.

Uniform Load.—Homogeneous Beam, Fig. 39.—

$$E I \frac{d^2 y}{dx^2} = M = \frac{1}{2} pl x - \frac{px^2}{2} - \frac{1}{14.2} pl^2$$

Integrating,

$$E I \frac{dy}{dx} = \frac{1}{4} pl x^2 - \frac{1}{6} px^3 - \frac{1}{14.2} pl^2 x + C_1$$

Integrating,

$$E I y = \frac{1}{12} pl x^3 - \frac{1}{24} px^4 - \frac{1}{28.4} pl^2 x^2 + C_2$$

But $x = \frac{1}{2} l$, and constants of integration, $C_1 = C_2 = 0$,

$$\text{or } E I y = \frac{1}{96} pl^4 - \frac{1}{384} pl^4 - \frac{1}{113.6} pl^4 = - \frac{4\,608}{43\,622} pl^4$$

$$y = - \frac{1}{910} \frac{pl^4}{E I} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Third-Point Loads.—Homogeneous Beam, Fig. 40.—

$$E I \frac{d^2 y}{dx^2} = M = \frac{W}{2} x - \frac{W}{2} \left(x - \frac{1}{3} l \right) - \frac{1}{12} W l$$

Integrating,

$$E\,I\,\frac{dy}{dx} = \frac{1}{12}\,W\,l\,x + C_1$$

Integrating,

$$E\,I\,y = \frac{1}{24}\,W\,l\,x^2 + C_2$$

But $x = \frac{1}{2}\,l$ and $C_1 = C_2 = 0$

or $$y = \frac{1}{96}\,\frac{W\,l^3}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

From Equations (7) and (8), $K_1 = \left(\frac{1}{96} - \frac{1}{910}\,\frac{W'}{W}\right)$ ..... (9)

The value for K_2 is obtained from the bending moment formulas previously derived; W' = dead weight of slab.

Simple Span Test.

Uniform Load.—Homogeneous Beam. Fig. 41.—

$$y = \frac{5}{384}\,\frac{pl^4}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and $$M = \frac{1}{8}\,pl^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Third-Point Loads.—

$$y = \frac{23}{1\,296}\,\frac{W\,l^3}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$M = \frac{1}{6}\,W\,l \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

From Equations (1) and (3),

$$K_1 = \left(\frac{1}{56.4} + \frac{1}{76.8}\,\frac{W'}{W}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

TABLE 15.—Bending Moment Factors.

Span and end condition.	Reinforcement.		Load, in Pounds.		Equivalent uniform load. $\frac{4}{3}W + W'$, in pounds.	Analysis of Slab at Deformation Within First Stage.								
						Measured stress, in pounds per square inch.		Computed stress.			Deflection, in inches.		Derived B. M. coefficient.	
								Common theory.		Deflection.			Method.	
	Nominal area, in square inches.	Effective area, in square inches.	W	W'		Steel.	Concrete.	Steel	Concrete.	Steel.	Measured.	Computed.	1	2	3	4
C 7-ft., No. 1	0.202	0.173	2 125	481	3 313	3 000	117	35 500	570	3 690	0.045	0.040	1/78	1/190	1/155	1/21
C 7-ft., No. 2	0.202	0.173	2 400	473	3 673	9 000	520	41 000	675	.....	0.09	0.17	1/21	1/72	....	1/21
C 7-ft., No. 3	0.202	0.173	2 200	481	3 414	12 000	438	36 000	580	6 720	0.13	0.17	1/22	1/49	1/88	1/21
7-ft., No. 1	0.202	0.173	880	478	1 770	7 500	537	35 900	413	927	0.10	0.15	1/7	1/41	1/33	1/8
7-ft., No. 2	0.202	0.173	1 268	525	2 215	12 800	772	49 200	800	1 935	0.13	0.21	1/7	1/26	1/17	1/8
7-ft. No. 3	0.202	0.173	1 056	501	1 910	5 000	553	41 250	666	.....	0.04	0.13	1/9	1/62	....	1/8
C—6-ft., No. 1	0.187	0.173	4 300	386	6 126	14 050	475	60 500	1 134	24 060	0.24	0.17	1/37	1/69	1/41	1/20
C 6-ft., No. 2	0.187	0.173	4 800	393	6 793	8 325	660	65 800	1 205	16 050	0.21	0.17	1/29	1/130	1/60	1/20
C—6-ft., No. 3	0.187	0.173	4 800	423	6 823	4 200	590	59 800	988	.....	0.07	0.11	1/27	1/200	....	1/20
6-ft., No. 1	0.187	0.173	1 375	400	2 235	4 010	240	42 700	759	.....	0.01	0.06	1/26	1/85	....	1/8
6-ft., No. 2	0.187	0.173	1 585	436	2 561	9 500	476	43 300	695	.....	0.04	0.11	1/12	1/37	....	1/8
6 ft., No. 3	0.187	0.173	1 855	401	2 876	5 500	280	54 300	975	12 000	0.11	0.04	1/28	1/79	1/38	1/8

TABLE 16.—Modulus of Elasticity by Flexure of Plain Slabs.

W', Dead load, in pounds.	W, Concentrations, in pounds.	Deflection, in inches.	Bending moment, in inch-pounds.	K, in pounds per square inch.	E, in pounds per square inch.
No. 1.—Moment of Inertia = 151.8 in.^{4} Section Modulus = 69.5 in.^{3} K = Extreme Fiber Stress.					
360	1 100	0.010	13 700	197	3 480 000
360	1 200	0.012	14 700	212	3 080 000
360	1 300	0.015	15 700	226	2 630 000
360	1 400	0.019	16 700	240	2 210 000
360	1 500	.....	17 700	254	.....
No. 2.—Moment of Inertia = 140.8 in.^{4} Section Modulus = 66.1 in.^{3}					
350	1 100	0.0085	13 600	206	4 360 000
350	1 200	0.0115	14 600	221	3 450 000
350	1 300	0.014	15 600	236	3 020 000
350	1 400	0.0175	16 600	251	2 580 000
350	1 500	0.0275	17 600	266	1 740 000
350	1 510	.....	17 700	268	.....

TABLE 17.—Design Table for Percentage of Reinforcement from 0.1 to 1.0 Per Cent.

b = Breadth of slab;

d = Depth to steel;

k = Ratio of depth of neutral axis to depth of steel;

j = Ratio of effective depth to depth of steel;

$M_R{}^c$ = Resisting moment of section, based on concrete stress;

$M_R{}^s$ = Resisting moment of section, based on steel stress;

$M_R{}^c = (150\ k\ j)\ bd^2 \ldots\ldots = K_c\ bd^2$;

$M_R{}^s = (16\,000\ j\ p)\ bd^2 \ldots\ldots = K_s\ bd^2$.

Percentage.	k	j	K_c	K_s
0.1	0.22	0.93	31	15
0.15	0.26	0.91	35	22
0.18	0.28	0.91	38	26
0.19	0.29	0.90	39	27
0.20	0.29	0.90	39	29
0.21	0.30	0.90	41	30
0.22	0.30	0.90	41	32
0.24	0.31	0.90	42	35
0.25	0.32	0.89	43	36
0.26	0.32	0.89	43	37
0.27	0.33	0.89	44	38
0.28	0.33	0.89	44	40
0.30	0.34	0.89	45	43
0.32	0.35	0.88	45	45
0.33	0.35	0.88	45	47
0.35	0.36	0.88	48	49
0.36	0.36	0.88	48	51
0.38	0.37	0.88	49	53
0.40	0.38	0.87	50	56
0.5	0.42	0.86	55	69
0.6	0.45	0.85	57	82
0.7	0.47	0.84	59	94
0.8	0.49	0.84	62	108
0.9	0.51	0.83	64	119
1.0	0.53	0.82	65	131

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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HUACAL DAM, SONORA, MEXICO.

Discussion.*

By Messrs. L. J. Mensch† and H. Hawgood.‡

Mr. Mensch.

L. J. Mensch,§ M. Am. Soc. C. E. (by letter).—The problem of the arched dam is statically indeterminate, and the stresses can only be found by taking the deformation of the arch into consideration.

Notation.—

Let w = the water pressure, in pounds per square foot;
H = the height of the dam, in feet;
h = the depth of water above the section in consideration, in feet;
r = the outside radius, in feet;
Δr = the change of radius due to forces acting on the dam;
d = the thickness of the arch, in feet, at any section;
d' = the thickness of the arch, in feet, at the base;
I = the moment of inertia of any section, in feet;
I' = the moment of inertia of the section at the base, in feet;
L = the length of the arch in feet;
f = the versed sine of the arch;
E = the modulus of elasticity of the masonry per square foot, in pounds, $= 288 \times 10^6$;
T = the thrust of the arch, in pounds per linear foot in depth;
G = the weight of a slice of the dam 1 ft. long;
n = the distance of the down-stream face of the arch to the intersection of the resultant of forces with the base, in feet;

* Continued from August, 1914, *Proceedings*.
† This is a continuation of the discussion by Mr. Mensch.
‡ Author's closure.
§ Chicago, Ill.

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A' = the work of the interior forces, in foot-pounds per linear foot of the height of the dam, when the structure deforms as an arch only;

A'' = the work of the interior forces when the arch deforms as a girder only;

A = the work of a slice of the arch 1 ft. long and H feet high when the dam deforms as a cantilever only;

P = the force, in pounds per linear foot of the height of the dam, due to arch action only;

M = any moment, in foot-pounds;

S = any stress, in pounds per square foot;

t = a change in temperature, here assumed to be 50° Fahr.;

c = the coefficient of expansion, 0.0000055;

$c.t.$ = the total relative expansion or contraction, equal to $\frac{1}{3\,640}$;

x and y = co-ordinates from a point of origin identical with the left abutment;

dx, dy, ds = increments of x, y, and the length of the arch;

$\Delta x', \Delta y', \Delta \rho'$ = the changes of the ordinates of any special point, x', y', and of the angle, ρ', which the tangent at x', y' forms with a line connecting the two abutments.

For curved beams, where the radius is from five to ten times larger than the depth of the beam, the textbooks give the following equations for the change of the co-ordinates of any point, x', y', of the arch when P is a force normal to any section, x, y, and M is the bending moment about x, y,

$$\Delta x' = \int_0^{x'} \frac{P\,dx}{E\,F} + \int_0^{x'} \frac{M\,ds}{E\,I} - \int_0^{x'} \frac{M\,y\,ds}{E\,I} \quad \ldots\ldots\ldots (1)$$

$$\Delta y' = \int_0^{x'} \frac{P\,dy}{E\,F} - x' \int_0^{x'} \frac{M\,ds}{E\,I} + \int_0^{x'} \frac{M\,x\,ds}{E\,I} \quad \ldots\ldots\ldots (2)$$

$$\Delta \rho' = \int_0^{x'} \frac{M\,ds}{E\,I} \quad \ldots\ldots\ldots (3)$$

Where the radius of the curved beam is about twice the depth of the beam, the foregoing equations give values about 10% larger than the exact formulas.

The writer also makes the customary assumption that the abutments are fixed and do not admit of any displacements or change of angle of the arch at the abutment.

Most arched dams subtend an angle of about 90°; the writer has shown* that an angle of 120° is the most favorable one. For simpli-

* "Reinforced Concrete Pocket Book."

fication, all the following investigations will relate to these two angles only. Mr. Mensch.

Influence of a Change of Temperature on the Arch When the Reservoir is Empty.—For a fall of temperature, for example, the arch has a tendency to shorten its length and span, but this is prevented by a pull or negative thrust from the abutments and by the moments, Ma, exerted by the abutments on the arch in order to keep the angles at these places from changing. No forces normal to the chord can be exerted by the abutments, because there are no exterior forces acting on the dam which can hold them in equilibrium. From Fig. 22 it follows that the moment about any point, x, y, is

$$M = T\,y - Ma \qquad (4)$$

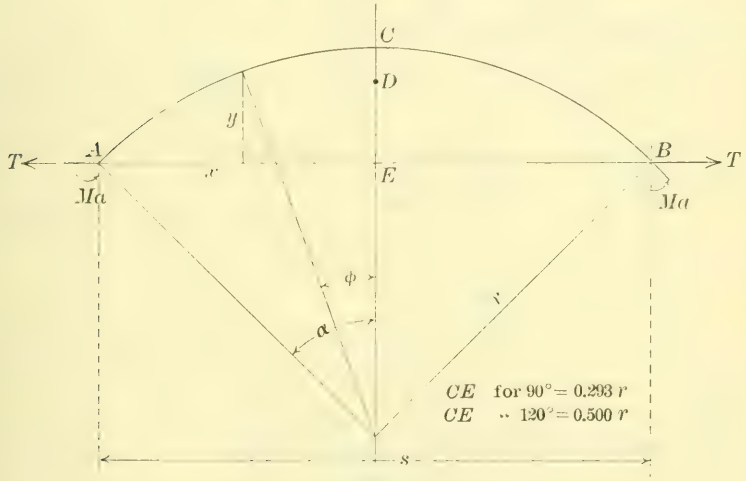

FIG. 22.

As the assumption is made that the abutments are fixed and do not allow any change of angle between them, it follows from Equation (3) that

$$\int_0^L \frac{M\,ds}{E\,I} = 0.$$

On account of the symmetry of the arch and the loading,

$$\int_0^L \frac{P\,dy}{E\,F} = 0.$$

hence Equation (2), taken between the limits, 0 and L, reduces to

$$\int_0^L \frac{M\,x\,ds}{E\,I} = 0,$$

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or, on account of Equation (4),

$$\int_0^L \frac{T\,y\,x\,ds}{E\,I} - \int_0^L \frac{Ma\,x\,ds}{E\,I} = 0,$$

which condition is fulfilled when

$$Ma = T\,\overline{D\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

when $\overline{D\,E}$ is the distance of the center of gravity of the arch from $A\,B$. Equation (1) now becomes

$$\Delta x' = \Delta s = \int_0^L \frac{P\,dx}{E\,I} - \int_0^L \frac{M\,y\,ds}{E\,I}$$

$$= \int_0^L \frac{P\,dx}{E\,I} - \int_0^L \frac{T\,y^2\,ds}{E\,I} + \int_0^L \frac{\overline{D\,E}\,T\,y\,ds}{E\,I}.$$

E, F, I, $\overline{D\,E}$, and T are constants; hence

$$\Delta s = \frac{1}{E\,F}\int_0^L P\,dx - \frac{T}{E\,I}\int_0^L y^2\,ds + \frac{\overline{D\,E}\,.\,T}{E\,I}\int_0^L y\,ds \ldots . (6)$$

The integration is facilitated by using polar co-ordinates, as shown in Fig. 22.

$$x = r\,(\text{sin. } \alpha - \text{sin. } \rho)$$
$$y = r\,(\text{cos. } \rho - \text{cos. } \alpha)$$
$$dx = -\,r\,\text{cos. } \rho\,d\,\rho$$
$$ds = -\,r\,d\,\rho$$
$$P = T\,\text{cos. } \rho$$
$$s = 2\,r\,\text{sin. } \alpha$$
$$\Delta s = c\,.\,t\,.\,s\,. = \frac{1}{3\,640}\,s.$$

It is easily found that $\overline{D\,E} = 0.193\,r$ for 90°;

and $\overline{D\,E} = 0.323\,r$ for 120°.

Hence, for an arch subtending 90°, Equation (6) becomes

$$-c\,.\,t\,.\,s\,. = -\frac{2\,T\,r}{E\,F}\int_{45}^{0} \text{cos.}^2\,\rho\,d\,\rho + \frac{2\,T\,r^3}{E\,I}(\text{cos. } \rho - \text{cos. } \alpha)^2\,d\,\rho$$

$$- \frac{2\,T\,0.193\,r^3}{E\,I}(\text{cos. } \rho - \text{cos. } \alpha)\,d\,\rho.$$

The integrals may be found in any handbook on calculus, and

$$-\frac{1}{3\,640}\,2\,r\,\text{sin. } \alpha = -\frac{2\,T\,r}{E\,F}\left[\frac{1}{4}\,\text{sin. } 2\,\rho + \frac{\rho}{2}\right]_{45}^{0}$$

$$- \frac{2\,T\,r^3}{E\,I}\left[\frac{1}{4}\,\text{sin. } 2\,\rho + \frac{\rho}{2} - 2\,\text{cos. } \alpha\,\text{sin. } \rho + \rho\,\text{cos.}^2\,\alpha\right]_{45}^{0}$$

$$- \frac{2\,T\,r^3}{E\,I}\,0.193\left[\text{sin. } \rho - \rho\,\text{cos. } \alpha\right]_{45}^{0}$$

$$-\frac{1}{3\,640}\,2\,r\,(0.707) = \frac{T\,r}{E\,F}\,1.285 + \frac{T\,r^3}{E\,I}\,(0.07) - \frac{T\,r^3}{E\,I}\,(0.0586) \quad \ldots (7)$$

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$$F = d,\ I = \frac{d^3}{12},\ \text{and}\ T = \frac{816\,000}{1 + 9.4\,\frac{d^2}{r^2}}\,\frac{d^3}{r^2}.$$

The moment at the crown is

$$0.100\,r\,T = 81\,600\,\frac{d^3}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \quad \ldots (8)$$

The moment at the abutment is

$$0.193\,r\,T = 157\,500\,\frac{d^3}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \quad \ldots (9)$$

The moment of resistance of a ring of the arch, 1 ft. high and d feet thick, equals $\frac{d^2}{6}$, and the stresses in the extreme fibers of the crown section equal

$$489\,600\,\frac{d}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \quad \ldots (10)$$

And those at the abutments equal

$$945\,000\,\frac{d}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \quad \ldots (11)$$

For an arch subtending an angle of 120°, we obtain, similarly,

$$T = \frac{228\,500}{1 + 2.47\,\frac{d^2}{r^2}}\,\frac{d^3}{r^2} \quad \ldots (12)$$

And the stresses in the extreme fibers at the crown equal

$$\frac{242\,300}{1 + 2.47\,\frac{d^2}{r^2}}\,\frac{d}{r} \quad \ldots (13)$$

The stresses in the extreme fibers at the abutments equal

$$\frac{442\,800}{1 + 2.47\,\frac{d^2}{r^2}}\,\frac{d}{r} \quad \ldots (14)$$

These stresses are often very high, especially in thick dams, and are increased considerably by the shrinkage of the concrete, which may amount to $\frac{1}{2\,500}$ of the length in a year. The assumed shortening of the arch under a fall of 50° Fahr. was $\frac{1}{3\,640}$ of its length, hence the shrinkage stresses amount to nearly 1½ times as much. There is actu-

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ally no way of preventing cracks, and the best practice is to provide vertical joints along radial planes at distances of about 50 ft., which will have the effect of concentrating the cracks in the least dangerous places, and to make the cracks between the joints very fine.

The Dam as a Pure Arch Structure.—Assuming the arch to be a portion of a thin cylinder acted on by exterior water pressure, it is known that the diminution of the radius,

$$\Delta r = \frac{w\,h\,r}{d\,E}\,\frac{2\,\pi\,r}{2\,\pi} = \frac{w\,h\,r^2}{d\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The arch would take the position, *A′ C′ B′*, if not restrained. It can be imagined that the restraint will cause the arch first to take the position, *A″ C″ B″*, and then, through a negative thrust, *T*, and a moment, *Ma*, to take the position, *A F B*.

This behavior is commonly called the shortening of the arch, and represents the secondary action of the arch as a common girder, fixed at its ends, due to the deflection of the arch, and is practically identical with that caused by a fall of temperature; the unknown value, *T*, can be obtained by substituting in Equations (6) and (7), $2\,\Delta\,r \sin.\,\alpha$ for $\Delta\,s$.

Hence, $1.414\,\Delta\,r = \frac{T\,r^3}{E\,I}\,0.0114\,\left(1 + 9.4\,\frac{d^2}{r^2}\right)$ for a 90° arch.

Substituting for $\Delta\,r$ the value in Equation (15), and for I, $\frac{d^3}{12}$,

$$T = 10.33\,\frac{d^2\,w\,h}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The moment due to the shortening of the arch at the crown

$$= 0.100\,Tr = 1.033\,\frac{d^2\,w\,h}{1 + 9.4\,\frac{d^2}{r^2}}.$$

The moment due to the shortening of the arch at the abutments

$$= 0.193\,Tr = 1.95\,\frac{d^2\,w\,h}{1 + 9.4\,\frac{d^2}{r^2}}.$$

The extreme fiber stresses due to shortening at the crown are obtained by dividing the moment by $\frac{d^2}{6}$,

$$6.2\,w\,h\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}}.$$

And the extreme fiber stresses due to shortening at the abutments,

$$11.7\,w\,h\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}}.$$

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The direct compressive stresses from the thrust due to water pressure are

$$w\,h\,\frac{r}{d}.$$

In most of the dams cited by the author, $\frac{r}{d}$ is about 20, hence the maximum compressive stresses due to arch action and the shortening of the arch are from 30 to 60% greater than those ordinarily assumed. In the Huacal Dam, $\frac{r}{d} = 6$, hence the maximum stresses are at least double those assumed.

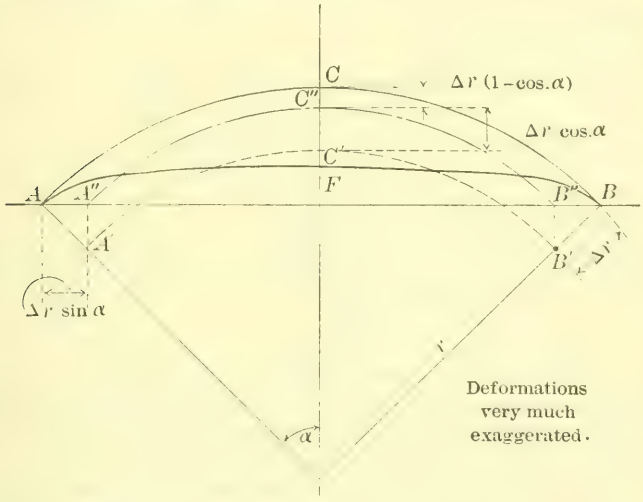

FIG. 23.

For arches subtending an angle of 120°, the maximum fiber stresses from the shortening of the arch at the crown are

$$3.072\ w\ h\ \frac{1}{1 + 2.47\ \frac{d^2}{r^2}}.$$

And the maximum fiber stresses at the abutment

$$5.61\ w\ h\ \frac{1}{1 + 2.47\ \frac{d^2}{r^2}}.$$

The distance, $C''\ F$, in Fig. 23, can be found from Equation (2) by omitting the second expression, which vanishes, as heretofore explained.

$$\Delta\ y = \int_0^{\frac{L}{2}} \frac{P\ dx}{E\ F} - \int_0^{\frac{L}{2}} \frac{M\ x\ ds}{E\ I}.$$

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Introducing again polar co-ordinates, and $M = T \cdot y - Ma$.
And, for 90°, $Ma = 0.193\ T \cdot r$

$$\Delta y = C'' F = -\int_{45}^{0} \frac{T r}{E F} \cos.^2 \rho \, d\rho$$

$$-\int_{45}^{0} \frac{T r^3}{E I} (\cos. \rho - 0.9)(\sin. \alpha - \sin. \rho)\, d\rho$$

$$C'' F = \frac{T r}{E F}(0.6425) - \frac{T r^3}{E I}(0.014) = 0.014 \frac{T r^3}{E I}\left[1 + \frac{I}{r^2 F}\frac{0.6425}{0.014}\right]$$

The value of T is given by Equation (16), and, neglecting the expression containing $\frac{I}{r^2 F}$,

$$C'' F = 10.33 \frac{d^2}{r}\frac{w h}{E I} r^3 (0.014) = 1.735 \frac{w h r^2}{E d} = 1.735 \Delta r$$

and, from Fig. 23, it follows directly

$$C C'' = 0.293 \Delta r$$

or $C F$ equals the total deflection for 90°

$$= 2.028 \Delta r \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Similarly, the total deflection for 120°

$$= 1.66 \Delta r.$$

Work of Deformation in an Arch Structure.—The work performed by the interior forces of an arch may be divided into two parts, namely, that performed by the direct compressive stresses, and that by the moments due to the shortening of the arch. The textbooks give the following formulas for the work of the interior forces of a prism subjected to compressive or tensile forces and for the work performed by a girder subjected to bending moments.

$$A' = \frac{L P^2}{2 E F} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

$$A'' = \int_0^L \frac{M^2 ds}{2 E I} \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

For an arch of 90°,

$$L = \frac{\pi}{2} r,$$

$$P = w h r,$$

$$F = d,$$

and

$$A' = \frac{\pi}{2} r \frac{w^2 h^2 r^2}{2 E d} = \frac{\pi}{4} \frac{w^2 h^2 r^3}{E d} \ldots\ldots\ldots\ldots (20)$$

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Let T from Equation (16) equal the thrust due to shortening of the arch, then the work due to shortening,

$$A'' = \int_0^L \frac{M^2\,ds}{2\,E\,I} = \int_0^L \frac{(T\,y - Mo)^2\,ds}{2\,E\,I}$$

$$= -2\int_{45}^{0} \frac{T\,r^3}{E\,I}(\cos.^2\rho\ 1.8 + \cos.\rho + 0.81)\,d\rho$$

$$A'' = 2\frac{T^2\,r^3}{E\,I}(0.00600) = 15.4\,\frac{w^2\,h^2\,r\,d}{1 + 9.4\,\frac{d^2}{r^2}} \quad \ldots\ldots\ldots(21)$$

and the total work

$$A' + A'' = A_1 = \frac{\pi}{4}\,\frac{w^2\,h^2\,r^3}{E\,d}\left[1 + 19.6\,\frac{d^2}{r^2}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}}\right] \quad \ldots(22)$$

For an arch of 120°, the total interior work

$$A_1 = \frac{\pi}{3}\,\frac{w^2\,h^2\,r^3}{E\,d}\left[1 + 4.55\,\frac{d^2}{r^2}\,\frac{1}{1 + 2.47\,\frac{d^2}{r^2}}\right] \quad \ldots\ldots\ldots(23)$$

Work of Deformation in a Cantilever Structure.—Assume a slice of the dam 1 ft. wide and h feet high, subjected to full water pressure. The moment about the base is

$$M = wh \times \frac{h}{2} \times \frac{h}{3} = \frac{wh^3}{6} \text{ in foot-pounds.}$$

and, according to Equation (19),

$$A_2 = \int_0^h \frac{M^2\,d\,h}{E\,I} = \int_0^h \frac{w^2\,h^6\,d\,h}{36\,E\,I} = \frac{w^2\,h^7}{252\,E\,I} \quad \ldots\ldots\ldots(24)$$

provided the thickness of the slice is constant from base to top. This is not the case, but it can be easily shown that the value of this work is affected comparatively little by the change of d, because most of the work is performed in the lower part of the cantilever, where the change of thickness is comparatively small. For example, the work performed in the lower fifth of the cantilever is

$$\int_{0.8h}^{h} \frac{w^2\,h^6\,d\,h}{36\,E\,I} = \frac{w^2\,h^7}{252\,E\,I}\,[1 - 0.21] = \frac{w^2\,h^7}{252\,E\,I}\,0.79,$$

or 79% of the total work; hence a comparatively small error is made when we assume I to be constant and write

$$A_2 = \frac{w^2\,h^7}{200\,E\,I} \quad \ldots\ldots\ldots(25)$$

The work performed for the total length of an arch of 90° is

$$\frac{\pi}{2}\,r\,A_2 = \frac{\pi}{2}\,\frac{w^2\,h^7\,r}{200\,E\,I} \quad \ldots\ldots\ldots(26)$$

Mr. Mensch. We will compare this value with the work in pure compression as given by Equation (20), which latter must be multiplied by h to obtain the value for the entire arch.

$$\frac{A_1}{A_2} = \frac{\text{Work of compression}}{\text{Work of cantilever}} = \frac{\pi}{4} \cdot \frac{\frac{w^2\ h^2\ r^3\ h}{E\ d}}{\frac{\pi}{400} \cdot \frac{w^2\ h^7\ r}{\frac{E\ d^3}{12}}} = 8.33\ \frac{d^2\ r^2}{h^4} \quad \ldots(26)$$

For the Huacal Dam, $d' = 13$ ft., $r = 76$ ft., or $r^2 d'^2 = 10^6$, and the work of compression equals the work of the cantilever action when the water stands 54 ft. above the base of the dam. When the water stands only 36 ft. above the ground,

$$\frac{A_1}{A_2} = \frac{8.33 \times 10^6}{168 \times 10^4} = 4.96,$$

hence the arch action can take up only 16% of the load on the dam. The value of A_2 of Equation (24) is only correct as long as the resultant forces strike the base within the middle third. Inasmuch as we cannot figure on any tensile stresses in the joint of the dam, quite another distribution of stresses takes place when the resultant comes nearer to the down-stream edge. The joints at the up-stream edge open up, and the compressive stresses will increase in a much greater ratio than the ordinary straight-line formula indicates.

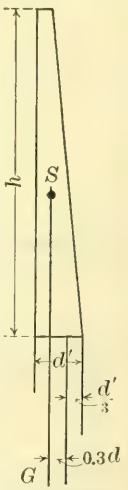

FIG. 24.

Fig. 24 shows that the maximum moment which the Huacal Dam can exert as a cantilever per linear foot equals the weight of the dam multiplied by 0.3 d'. The weight of the dam is about 127 500 lb., and the base is 13 ft., hence the maximum moment at which the resultant strikes the middle third is 510 000 ft-lb. As soon as this moment is exceeded, tensile stresses will appear at the up-stream base of the dam.

In non-reinforced dams the joints cannot be relied on to take the tensile stresses, and, as soon as the foregoing moment is exceeded, cracks will appear and allow the water to exert an upward pressure. In the extreme case, the resultant will strike the base near the down-stream edge, which requires a moment from the horizontal water pressure of approximately $G \times 0.6\ d' = 1\,020\,000$ ft-lb. In this case the joint will be open nearly the entire width of the base, and the upward water pressure will exert a moment of

$$62\tfrac{1}{2} \times h \times d' \times \frac{d'}{2} = 31.25\ h\ d^2 = 525\,000 \text{ ft-lb.}$$

It simply shows that after the resultant strikes the base at the edge of the middle third, the cantilever action in non-reinforced dams is practically exhausted.

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The height of water which will exert a moment of 510 000 ft-lb. is found from the following equation:

$$M = \frac{w\,h^3}{6}, \text{ or } h = \sqrt[3]{\frac{6\,M}{60}} = 36 \text{ ft.}$$

It will be assumed that the water stands 36 ft. above the base and that the portion of the dam above this level does not influence the action below.

If the dam acts as an arch only, the deflection in the center of the dam at the base would be given by Equation (17),

$$\frac{2.028\,w\,h\,r^2}{d'\,E} = 0.00716 \text{ ft.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

If the dam acts as a cantilever only, the deflection of the point 36 ft. above the base would be

$$\int_0^h \frac{M\,x\,dx}{E\,I} = \int_0^h \frac{w\,h^3\,.\,h\,d\,h}{6\,E\,I} = \frac{w\,h^5}{30\,E\,I} = \frac{12}{30}\,\frac{w\,h^5}{E\,d^3}$$

$$= \frac{12}{30}\,\frac{6.25 \times 36^5}{288 \times 10^6 \times 12.833^3} = 0.00256 \text{ ft.} \quad \ldots\ldots\ldots (28)$$

The line, $H\ M\ K$, in Fig. 25, shows the ideal deflection of the dam acting as an arch only when the water is 36 ft. above the base, and the line, $G\ L$, shows the ideal deflection of the dam acting as a cantilever, assuming that the upper portion of the dam is disconnected from the lower portion. Inasmuch as a portion of the water pressure is taken up by the arch action, the deflection of the cantilever must be less, and may be represented by the line, $L'\ M'\ G$. At any point, P, for example, the total water pressure represented by the deflection, $P\ O$, is divided into two parts, one which acts on the dam on the arch principle, represented by $P\ Q$, and one which acts on the dam on the cantilever principle, represented by $Q\ O$.

We have seen from the foregoing that in a pure arch structure the deflections are proportionate to $\frac{w\,h}{d}$, hence the water pressure at the base is directly proportionate to $G\ K$; at P they are directly proportionate to $\overline{P\ O} \times \frac{d}{d'}$; then that part of the pressure acting on the cantilever principle is directly proportionate to $\overline{Q\ O} \times \frac{d}{d'}$, and that part of the pressure acting on the arch principle is directly proportionate to $P\ Q\ \frac{d}{d'}$. The area, $G\ M'\ K'$, reduced by the proper $\frac{d}{d'}$, represents the sum of all the pressures acting in a down-stream direction, which try to overturn the

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dam as a gravity structure, the area, $G\ H\ L'$, reduced by the proper $\frac{d}{d'}$, represents the sum of all pressures acting on the arch principle; and the area, $H\ L'\ M'$, properly reduced, represents the reaction in the up-stream direction on a slice of the dam which helps to prevent overturning.

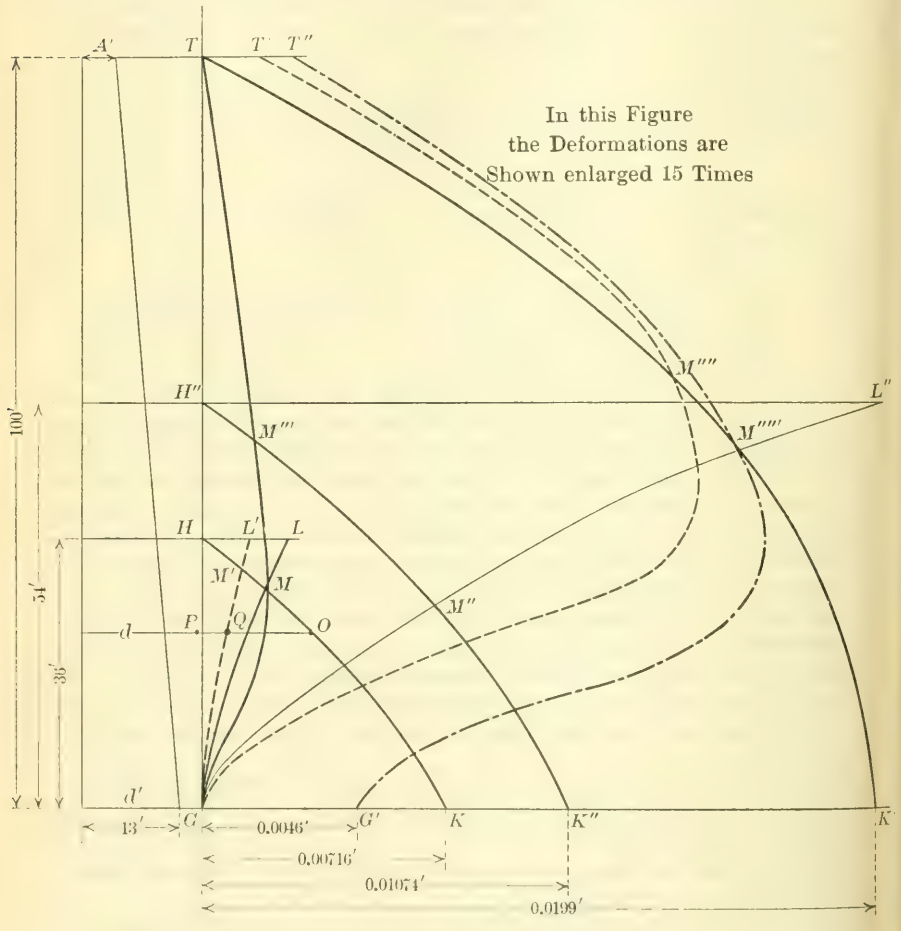

FIG. 25.

It is also clear that the maximum moment at the base of the dam, when the water is 36 ft. above it, is less than 510 000 ft-lb., and, therefore, at no point of the arch does the resultant of forces come outside of the middle third of any section. It will now be assumed that the water level is 54 ft. above the base of the dam. $H''\ M'''\ K''$ repre-

sents the deflection of the dam as a pure arch structure. $G\ M''\ L''$ would represent the deflection of the dam as a cantilever structure if a slice of the dam could sustain moments caused by the water pressure of 54 ft., which, however, is not the case, and would be possible only in heavily reinforced dams. Mr. Mensch.

It must be further considered that a slice of the dam does not really act as a pure cantilever; it really acts as a girder supported at the base against sliding by a horizontal up-stream reaction and by the bending moment of 510 000 ft-lb., and supported in the upper portion of the dam by the flexible support consisting of the top portion of the dam. The line, $T\ M'''\ G$, represents the deflection of the vertical slice of the dam. This, of course, is a mere assumption, and the line must fulfill the following conditions: First, that the bending moment of the area, $G\ M'''\ K''$, minus the moment of the area, $H''\ M'''\ T$, is not much larger than 510 000 ft-lb. Second, that the line of flexure assumed must be the same as that obtained by having the beam, $G\ T$, loaded by positive and negative forces represented by these two triangles.

It can be easily found that, also, in this case, the resultant forces at no point are outside of the middle third of any section.

It will now be assumed that the water level rises to the top of the dam; then the deflection of the dam as a pure arch structure is given by $T\ M''''\ K'''$. If the dam be heavily reinforced, both for bending moments and for shear, and well anchored in the rock, it can be established that the line, $G\ M''''\ T'$, would represent the deflection. In this case the area, $G\ M''''\ K'''$, properly reduced, amounts to a water pressure of 140 000 lb. Inasmuch as the weight of the dam is 127 000 lb., it is impossible, in a common non-reinforced structure of the dimensions of the Huacal Dam, that the friction is not overcome, and the structure will slide somewhere near its base. The line of flexure now will be somewhat as given by the dot and dash line, $G'\ M''''\ T''$. The area, $G'\ M''''\ K'''$, must represent a smaller pressure than about 60% of the weight of the dam, and the moments of the forces represented by the area, $T\ T''\ M''''$, will produce a larger bending moment than the dam as a gravity structure at a section about 32 ft. above the base can stand.

If the reinforcing in the Huacal Dam had been omitted, cracks probably would have shown at this point on the down-stream side, and in some points probably higher up.

Such cracks occurred on several of the dams built by Mr. Wade, in Australia. It must be noted that the movement of the base needs to be only very small, and is less than that caused in tangential directions by a fall of temperature or shrinkage of the concrete.

The foregoing conclusion disregarded the fact that vertical joints are often provided in the dam, or that cracks are formed by the com-

Mr. Mensch. bined action of a drop of temperature and the shrinkage of the concrete when the reservoir is empty.

The result is a considerable increase of the stresses due to the shortening of the arch, and a greater movement of the base of the arch when the reservoir is full. This, however, will not materially change the theory presented by the writer.

Mr. Hawgood. H. Hawgood,* M. Am. Soc. C. E. (by letter).—The details of the dams at Winchester, Ky., and Lewiston, Idaho, contributed by Mr. Sherman, make a valuable addition to the list of arch dams in Table 1. It does not appear, however, that the other dams cited by Mr. Sherman should be given place in this table, for they are not pure arch dams, to the listing of which that table is restricted.

The following data relating to an arch dam designed by Mr. G. O. Newman, Los Angeles, and constructed in 1913, may be added to the list:

Location, Triunfo Creek, Los Angeles County, California;
Up-stream face vertical, radius 287.9 ft.;
Down-stream face battered 3½ in. per ft.;
Crest length, 148 ft.;
Arc, 29° 36′;
Total height, 50 ft. above foundation;
Crest height above stream bed, 41 ft.;
Thickness at 41 ft. below crest, 15 ft. 6 in.;
Spillway, notch in dam crest, net length 80 ft., depth 5 ft.;
Overflow, January, 1914, 5 ft. 6 in. deep over crest, 10 ft. 6 in. over spillway; approximate discharge, 10 000 sec-ft.;
Stress at 46 ft. 6 in. below surface of flood overflow, 27 tons per sq. ft., or 373 lb. per sq. in.;
Material, concrete; coarse aggregate, a hard igneous rock, unclassified.

Mr. Sherman is entitled to thanks for his drawing of the forms used in building the Las Vegas Dam. Engineering papers, otherwise replete with details of design, are often lacking in information as to the manner of building. How to carry a design into effect is as important from the economic viewpoint as the design itself; sometimes more so.

In Table 9, Mr. Sherman notes the top width of the Huacal Dam as 3.5 ft., followed by an interrogation point. The width, or thickness, at Elevation 4 179.5, where the corbelling commences, is 44 in.; the figures are somewhat obscure in Fig. 7. This width is increased to 51 in. on top to provide a convenient footway.

* Los Angeles, Cal.

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At the present date, January, 1915, the Huacal Reservoir is full, and there has been an overflow discharge over the dam crest to a depth of 18 in. The structure is designed for an overflow of 4 ft. 6 in.

The subject of overflow is of more than passing interest, and it may be well to note that the La Grange Diversion Dam, on the Tuolumne River, California, has been subjected to overflow to a depth of 17 ft. The discharge corresponding to this depth is about 80 000 sec-ft. The La Grange Dam, completed in 1894, is reputed to be the highest overflow dam in the United States, if not in the world. The height of the up-stream face is 125 ft., and of the down-stream face 129 ft. It has a gravity section, with ogee profile, and was completed in 1894.

The writer appreciates Mr. O'Shaughnessy's criticism, but does not share with him any apprehensions as to greater risks sustained by an arch than by a gravity dam. Indeed, the arch type has fewer elements of danger than the gravity type. Soundness of foundation, essential for both, is essential to even a greater degree with the gravity than with the arch type. The danger of hydrostatic pressure tending to lift the dam, a serious danger to the gravity type, is a matter of no such moment to the arch. The sliding of a stratified foundation rock, as in the Austin, Tex., dam failure, would not occur under an arch type where the water thrust is transferred to the abutments. The thrusting apart of the flanking hills is inconceivable. The crushing of the arch itself is eliminated by the adoption of safe unit stresses. As to earthquakes, is it not true that a *temblor* sufficiently violent to wreck a structure having some degree of flexibility, such as possessed by an arch, would also wreck an inflexible gravity dam?

Mr. O'Shaughnessy has requested more information regarding the spillway gates. Each of the nine gates is 98 in. wide, with the central pivot $\frac{3}{4}$ in. to one side of the true center. The narrow leaf has a height of 4 ft. 6 in., and the wider one a height of 2 ft. 9 in. The pressure on the two leaves is in balance when water overflows the lower and wider leaf to a depth of 9 in. A further rise of water will bring excess of pressure on the higher and narrower leaf, and cause the gates to swing open.

The floods of this season were not quite of sufficient height to test fully the operation of the gates. They were successful as far as tried out.

The field of the arch dam is not confined to narrow gorges, witness the Barossa Dam, 472 ft. long, and the Parkes Dam, 540 ft. long. It has also an economic field in the case of a dam too long for the pure arch type by taking the place of the gravity section in the highest, and hence the most costly, part of such a dam, the gravity sections on each side serving as abutments. Multiple arches covering the entire

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length of the dam could be used in such a case. It would be unduly extending the scope of this paper, however, to take up the question of multiple arches.

During the discussion of the paper* by Mr. L. A. B. Wade, entitled "Concrete and Masonry Dam-Construction in New South Wales", it was brought out that the arch type required about the same volume of masonry as the gravity type when the radius of curvature, in feet, was somewhat more than twenty times the limiting stress, in tons per square foot. This approximation is useful as a preliminary test as to the suitability or otherwise of the arch for any particular location.

Theoretically, the radius necessitating the minimum quantity of masonry is that which makes the central angle of the arch about 133 degrees. Mr. Jorgensen, in his article, "Arch Dam Design",† goes into the subject at some length.

Mr. Wade expressed the same result by saying that the most economical radius was that which made the versed sine about one-third of the chord.

In a still simpler form, the theoretical economic radius is equal to the chord multiplied by 0.54.

The foregoing will serve to give the maximum and minimum radii within which the best one is to be sought. It will rarely happen in practice that the mathematical formula will control. The configuration of the rock walls which are to serve as abutments, and particularly the relation thereto of the resultant arch thrust, will usually exercise a strong influence on the selection of the radius best adapted to the location.

Replying to Mr. Jorgensen, the vertical reinforcement of old 12-lb. rails was introduced for two reasons: First, to take up temperature stresses, which, under the conditions, may be quite material because the down-stream face is exposed during by far the greater part of the day to a hot sun, followed by cool nights, and the up-stream face is always in the shade and more or less covered by water. Secondly, the rails formed stout stanchions to which the horizontal reinforcement could be wired most conveniently at the desired spacing, and in other ways they were useful as anchorages during construction operations.

Mr. Mensch, in adhering to the theory of a dam connected to a rigid base, misinterpreted the writer's statement that "Deformation of the rock undoubtedly takes place, and, such being the case, the arch action takes place in some unknown degree from the very base of the dam." Deformation was here used, and obviously so, in the sense of elastic deformation, and not in the sense of dislocation, as interpreted by Mr. Mensch.

* *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 1.

† *Engineering News*, July 25th, 1912.

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The writer is of the belief that there is no substance in Nature known to engineers, which is absolutely incompressible, inextensible, or void of elastic properties, and, such being the case, it must then follow that the rock on which the dam is built is deformed by the stresses to which it is subjected, just as the same rock, if wrought into the voussoirs of an arch bridge, would contract and expand with every change of load and temperature, as shown by the vertical movement of the arch crown.

Mr. Mensch's treatment of the theoretical stresses in arch dams is a contribution to the mathematics of the subject which will be appreciated by the members. Such arguments are most useful, but their field is suggestive rather than conclusive. The wide difference between theoretical and actual results is strikingly illustrated by comparing the theoretical deflection curves, shown in Fig. 25, with the actual deflections of the Barren Jack Dam, shown in Fig. 6.

One weakness of attempts at mathematical solution is the lack of real knowledge as to the true value of the modulus of elasticity, and the impossibility of one value applying to all parts of a structure changing from concrete thick with plum stones to concrete without any, together with all the variations in manner and perfection of mixing and placing. A recognized value of the modulus for 1:2:4 concrete is 3 000 000 per sq. in., or 432×10^6 per sq. ft. Mr. Mensch uses 288×10^6, and undoubtedly is not without authority for so doing. The value to be adopted is purely a matter of opinion, and the conclusions will be as divergent as the opinions.

Another weakness is the common assumption that the dam is held absolutely rigid at its base. Such an assumption is contrary to fact.

The typical foundation of a dam is a trench in the country rock, filled with concrete, on which the dam proper rests and to which it is commonly keyed. The concrete base has essentially the same modulus of elasticity as the dam above it, and will yield proportionately to the thrust brought on it by the water thrust on the dam. The elastic deformation will not be infinitesimal, but appreciable, even if small, and thus the theory of a rigid base is ended. Again, the rock itself is not without elasticity, and when it is considered that most rock, even when structurally strong and sound, is possessed of microscopic seams, the closing of these, under great pressure, together with the compression of the solid parts themselves, may well result in appreciable deformation which is as infinitesimal as Mr. Mensch would have us believe.

Again, the vertical pressure of the dam, as the French engineer, M. H. Billet, pointed out, produces a transverse dilation in the dam and its base.

All these things, together with other unknown conditions, render any mathematical conclusions but conjectural.

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The question as a whole is complex. Elements such as the vertical and lateral deformation of the bed-rock above the dam under its burden of superimposed water help to increase the base deformation in the down-stream direction, with opposing forces, on the other hand, tending to counteract. A complete analysis of the subject is impossible, there being too many unknown factors. The writer's object at this time will have been accomplished if sufficient has been introduced to indicate a reason for thin arch dams not cracking at their bases, as would inevitably take place if the cantilever action were in reality as great as we have been led to suppose on the rigid base theory.

The theory that the lower part of the dam is held fast in a vise of an absolutely unyielding, rigid nature cannot be logically deemed otherwise than an arbitrary assumption repugnant to the facts.

The theory that the rock base of a dam is deformed within its elastic limits under the stresses brought on it is in harmony with the laws of Nature, so far as known.

In discussing the deflection of the Huacal Dam, Mr. Mensch, in Equation 27, assumes the contained angle of the dam at its base to be 90°; this is not correct, the actual angle is 52°, and the deflection at that point would be about 25% less than that given by Equation 27, and other corrections will ensue.

Mr. Mensch speaks of possible maximum stresses in the Huacal Dam as at least double those ordinarily assumed, and in most of the other dams on the list as from 30 to 60% greater. This statement unqualified might inadvertently create the erroneous impression that the Huacal Dam was subjected to unusual stresses, whereas the contrary is really the case, that is, the Huacal Dam stress is 16 tons $\times$ 2 = 32 tons: that of the other dams is 28 tons $\times$ 1.5 = 42 tons.

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PAPERS AND DISCUSSIONS

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SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

Discussion.*

By Clarence T. Johnston, M. Am. Soc. C. E.†

Mr. Johnston.

Clarence T. Johnston,‡ M. Am. Soc. C. E. (by letter).—When we view land, there are two principal features of topography which impress us. The first is area, or the horizontal projection of the surface of the ground. The engineer measures this; the Government grants title to it; and private parties buy and sell it. Then there is elevation, which gives life and profile to landscape. This may be called the vertical projection of the ground surface. The first feature is stable, there is only one horizontal projection. The second feature depends on the location of the observer. Every time he changes his position, the scene changes. If altitude did not exist, there would be no scenery, or if there were, it would be monotonous. People who live in a flat country always lack some sources of inspiration. The vertical projection of land is of special public interest. The public does not part with this when title is given to horizontal areas. It cannot be bought and sold, because it supports art, beauty, sentiment, and fundamental principles. Private parties may change the surface of the ground in detail, shifting the vertical elements slightly, and still not transgress on the rights of those who appreciate and enjoy gifts which naturally belong to all.

It is because of the vertical component in landscapes that streams flow and lakes exist. Lakes form because a contour—a line of equal elevation—is continuous on the surface of the ground; the lands within the boundary of this contour being lower, water collects. Streams flow because the ground over which the water runs, has a constantly

* Continued from December, 1914, *Proceedings*.

† Author's closure.

‡ Ann Arbor, Mich.

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decreasing elevation, from the source of the river to the sea. As a stream is followed toward its source, it breaks up into innumerable tributaries, and these continue to sub-divide until the channels are almost indiscernible. This stream, its tributaries, and the many small channels, constitute a natural drainage system. Where a country is blessed with ample slope, that is, where the second feature of topography is represented in a valuable way, drainage is generally natural, and Man is relieved of much labor and responsibility. Where this feature of topography is wanting, in some degree, he must do more for himself. Streams and lakes, therefore, are of public character by decree of Nature. The vertical element of topography prevails throughout the drainage basins of streams, and it is present in and along their channels. Where rapids or falls occur, this component is manifested in a striking way. Without analyzing the principles on which the action has been taken, the public has preserved waterfalls, in some measure, for public benefit. The vertical element of topography in the channels of streams has an influence on irrigation, navigation, and the development of power, all, shall we say, matters of public concern. As the land to the right and left of the channel of the stream is investigated, the vertical element of the topography becomes more prominent and striking. There is nothing in Nature to indicate that an owner of riparian lands is more interested in the stream—except in so far as proximity is concerned—than is the owner of lands more remote from the river. The river serves both in matters of drainage, navigation, and possibly irrigation. It should furnish both with power and with water for domestic purposes, in so far as these may be needed. Nature, evidently, did not suggest the doctrine of riparian rights to Man.

When the Courts apply the common-law doctrine to streams and lakes, they go beyond legislative halls and the schools of Nature to secure the rules which govern. The doctrine of riparian rights could not live in the atmospheres of modern Governments were it not changed frequently in efforts to meet new conditions. The Court assumed legislative authority, in a limited way, when the doctrine was introduced, and each time new constructions are made, the judiciary invades the field of the lawmaker. When a Court finds the construction which it approves, a decree is issued; then another difficulty is presented. Because legislation, produced in this way, is not framed on broad lines, no administration is available to carry the rules of the Court into effect. The Courts have not hesitated when confronted by a serious obstacle of this kind. They appoint Court commissioners, or other officers, and take responsible charge of the administration. Since the Constitution was ratified, therefore, the judicial branch of the Government has been able to maintain its own exclusive field and to grasp something in both the legislative and executive departments.

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The definition of the doctrine of riparian rights, given by Mr. Bellamy,* is almost identical with that quoted by the writer.† The case, Mason *v.* Hill,‡ decided in 1833, records an epoch in the development of the doctrine. Prior to this decision there seems to have been some tendency, even in England, to give weight to priority. It breaks down any further claim in that direction. As early as 1805 Lord Ellenborough said:

"The general rule of law as applied to this subject is that, independent of any particular enjoyment used to be had by another, every man has a right to have the advantage of a flow of water in his own land without diminution or alteration."

The most notorious case in the United States, where the riparian doctrine has been attacked, is that of Lux *v.* Haggin.§ In this case, the conflicting claims of riparian owners and irrigators were brought to the attention of the Court. The decision was favorable to the riparian proprietors in the Lower Court, and an appeal was taken to the Supreme Court of the State. In the latter Court seven judges solemnly listened to testimony which revealed a war between the ancient and the modern, the impractical and the practical, the common law and statutory provisions pointing to something better. Four of these judges upheld the decision of the Lower Court, the principal point being that the Court had no power to modify the common law. As the common-law doctrine had been manufactured by Courts and repeatedly construed and misconstrued by them, one can with difficulty understand their caution on this occasion. The dissenting opinion, written by Judge Ross, is an able document considering that it was prepared thirty years ago when little was known, in the United States, of principles relating to the administration of streams. He defines the riparian doctrine as follows:

"By the common law, the water flowing in a river or stream, or down a cañon or ravine, could not be acquired by appropriation, and must continue to flow in its natural channel undiminished in quantity and unaffected in quality."

The judges who wrote the prevailing opinion quote several definitions which are almost identical in meaning. Although the case resulted in a victory for the riparian claimants, the Court did not attempt to determine the rights of the interested riparian proprietors.

If the riparian rights were limited to those which the owners of riparian lands naturally enjoy, very little trouble would follow its application. The owner of riparian land has advantages because of the proximity of the stream. It furnishes him a convenient source

* *Proceedings*, Am. Soc. C. E., for September, 1914, p. 2364.

† *Proceedings*, Am. Soc. C. E., for May, 1914, p. 1255.

‡ 5 Barn. and Adol. 1, 110 Eng. Reprint, 692.

§ 69 Cal., 255, at 398 *et seq.*, 10 Pac., 674.

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of water supply for all domestic purposes. It may be of value to him in the way of transportation. Domestic uses imply diversions from streams, yet these exist even in countries where the doctrine of riparian rights is presumed to be observed. A doctrine which permits (in practice rather than in theory) of a small diversion for a use which supports life, must permit of large diversions for the same use, or the doctrine fails. A doctrine which compels a town to pay tribute to an owner of riparian lands, regardless of beneficial use or preferred use, does not merit praise or support. Because the owner of riparian lands has natural advantages, other favors, of more weight and importance, should not be extended to him. The natural uses of the owner of riparian lands should be protected, but he should not be permitted to impose on others whose needs are as crying as his.

If engineers throughout the United States realized the problems that are presented in the administration of streams, more rapid progress would be made in the enactment of wise laws relating thereto. There are engineers who submit without question when they are told that this or that is the rule, and they are not inclined to study conditions elsewhere, apparently fearing that they might ultimately be led to recommend innovations which might not be appreciated, in the beginning at least, by employers. The engineer who specializes as an expert witness seldom progresses very far in the study of principles. Principles cannot be bought in the open market, and a discussion of principles by a witness is seldom countenanced. There are also engineers who constantly look forward to the creation of offices which may demand engineering talent, and any suggestion that principles be studied immediately reminds them that some officer must carry these principles into effect. Their usefulness in the great field of initiative is ruined, because they prefer to discuss machinery rather than purpose and design.

It is important, it will be admitted, whether or not an administration is to be under the control of National, State, county, or township officers. As water users, we would not worry greatly, regardless of the character of the administration, provided the principles, so necessary for a just division and distribution of the available water supply, were carried into effect without fear or favor. Further, after we have discovered all the principles that have been approved by the best authorities, we may be able to say what kind of an administration we need. It is possible that some or all of these principles should be incorporated in a National statute and be administered by National officers, as Mr. Lewis recommends; some may properly go to the States; others to the counties, and still others to lesser subdivisions of government, or to drainage basins, regardless of political lines. All kinds of interstate trouble may be imagined. We can go further and discover international problems. Thus far, we have had a little trouble

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on the Rio Grande and on the St. Lawrence. It might be best to have all streams under international control. However, we can well spend our time studying principles and, later, determine the kind of an administration we believe is necessary to carry them into effect. To have the administration first and an enunciation of principles later, might lead some to believe that the demand for office exceeds respect for principle. Generally, under these conditions, the principles which should govern are not separated from rules, which have no logical application, until long after the administration has made its way, limpingly and painfully, along a road that does not run parallel with efficient public service. It would be much better to have every State become interested in a discussion of principles, than to provide interstate commissions to regulate the use of water from streams. To establish these commissions now would simply mean that a deadlock would follow their first serious attempt to do business. Colorado, the procedure of which is based on principles not too plainly defined by law, is on a different basis from Kansas, which relies on the doctrine of riparian rights. Commissions made up of men from the two States would have to go back many centuries to secure common ground, and then they would find themselves unsupported by the laws or customs of either Commonwealth. A National commission might have an experience almost as discouraging.

Although all these troubles, and many others, would arise, and although a commission of engineers would have a rough way to travel, until wise principles were found and incorporated in the law, yet such a commission would be able to record some progress each day. Engineers understand the natural laws which govern streams. They know how the laws of Man must be framed in order that these natural laws be not violated. Bearing these things in mind, they are in position to study principles on which the man-made law should be based; so that the commission, wholly ignorant in the beginning of the principles which should govern, is to be preferred to the Court. The commission makes progress. The Court does not. However, the commission may fail while it is searching for principles. If within a reasonable time, the commission cannot announce some of the great principles for which it should stand, the law creating it may be repealed. The study of principles, therefore, should precede the creation of office.

The principle of priority is dangerous when it stands alone. When the doctrine of riparian rights was abrogated in some of the western States, the principle of priority was introduced, largely by the Courts, and it was unsupported by other principles for a long time. The Courts never discovered the principles that must go with and supplement the principle of priority in order to produce a thoroughly balanced administration. As soon as the principle of priority replaced the common-law rule, the Courts began to build on it a network of

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theory which brought almost as much injustice as was suffered under the original doctrine. Those whose business instincts had been sharpened by a somewhat rough experience, laid plans to seize streams under the protection of the new principle and by the consent of the administrators, the Courts. All kinds of testimony were brought to the Court. Though the Court kept the principle of priority in view, it did not know how to determine the dates which fixed the priority of rights. Time might begin to run when the prospective water user first thought of using the water, when he began a survey in the field, when he began or when he finished construction. The Courts did not know how to establish the limits of the rights to use water. In some cases, the right was limited by the claims made by the applicant in some office of public record; in others, by the size of the conduit which carried the water to the place where it was to be used. The Courts never made much progress when it came to limiting the right by the actual demands of the use, but were inclined to divide water among claimants and then view the rights thus defined as private property which might be bought and sold. These are only a few of the complications that arose, and yet the principle of priority was still observed. It was not supplemented by the few other principles that have already been briefly described. The complications which Mr. Horton describes, and his criticism of the principle of priority, as he sees it, are to be anticipated. He has enumerated* some of the things which give rise to litigation. These will be referred to by number and letter as he has designated them.

1*a*.—The determination of boundaries of "grants" has become necessary, it must be supposed, because a few people, presuming to represent the public, have assumed to give to private parties a title in and to the beds of streams and perhaps to the streams themselves. If all uses are to be protected, this kind of property cannot be taken from the public, and if there is a title resting anywhere, it at all times lies with the public. Even Courts, sooner or later, must come to this conclusion. If all users understood that the stream itself, including the channel and the water, is always public, there would be little occasion for a contest as to the location of such boundary lines as are generally concerned in litigation of this kind.

1*b*.—The difficulties which arise under this heading can be ascribed largely to the troubles which always prevail when private parties are given "rights" which concern, in any way, the "thread of the stream", "low-water mark", or "ordinary stage." The "high-water mark" should define the boundary between private and public property, and this should be located by the public, once and for all time. This at once assumes an administration. The administration is necessary to see to it that public interests are preserved and that private

* *Proceedings*, Am. Soc. C. E., for November, 1914, p. 2847.

uses do not conflict. Under these conditions, the public, only, would be interested in the "thread of the stream", "low-water mark", and "ordinary stage", and, in a few years, we would never hear the terms except when reviewing literature relating to public property and political boundaries. Mr. Johnston.

2*a*.—The problems presented by "flowage" are also simple when an administration is provided. With a central office of record and a knowledge of each stream, with laws requiring prospective water users to file applications and complete plans, all conflicts between power plants and other uses may be provided against before any injurious use is initiated.

2*b*.—When plans are submitted to the office of record, the public at once knows what lands must be flooded, and the parties proposing to bring about development are then obliged to deal with the land owners. Where public interests are involved, even in works proposed to be undertaken by private parties, a simple and direct method of condemnation should be provided. Should the parties not be able to proceed with construction within a reasonable time after these difficulties are removed, the permit issued to them should be canceled.

3*a*.—The diversion of water by cities, to the detriment of water-power plants, is now and will continue to be a source of litigation until a better understanding is reached. Mr. Horton is right when he says that cities have been "notoriously lacking in foresight in securing adequate sources of water supply." This is true, because the city represents the public, locally, at least. Our customs, if not our laws, generally favor the individual and the corporation as against the municipality. The city has been imposed on whenever the common-law doctrine has been invoked. Those doing business for the city have not had the best opportunity to gain the experience which would tend to develop and support an able administration. Every city has a small beginning. It is then in keen competition with private individuals. As some of these individuals are likely to be in partial charge of the municipal government, the city administration does not start with the best of prospects. Before the town has cast aside its swaddling clothes, the streets are largely occupied by things that belong to outsiders. The city fathers have one duty which appeals to them pre-eminently—the granting of franchises. Almost everything that concerns the convenience and the happiness of the people is given over to the management of private corporations, with the single and glaring exception of the sewer system. This is likely to be poor in design and construction, and to promise no profit in operation. It is, evidently, a good thing for the city to run, and future city fathers may obtain their best experience as administrators therein and thereby. It is not surprising that cities are "notoriously lacking in foresight in securing adequate sources of water supply."

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3*b*.—The troubles which may arise between users who divert water on opposite sides of a stream are probably numerous where an attempt is made to administer rivers and lakes under a doctrine which, in its fundamental form, makes diversions impossible. If the relative rights of each were to be defined by public authority, and each was required to live up to the terms of a specific agreement on record in the central administrative office, we would not anticipate much uncertainty as to the limits of rights and uses, and consequently we would not look for much, if any, litigation. The rule of priority, properly supported by other important principles, might be found to offer salvation under such conditions.

3*c*.—If the individual owners of a power plant, dam, or canal are inclined to quarrel, they should have permission to do so, as long as they conduct it as a family affair and make no attempt to stir up strife among those who have their rights defined and who, consequently, understand their relation to each other and to the public. It has been demonstrated, however, that where an administrative control is exercised, there is little cause for uncertainty, and uncertainties always lead to trouble. Often those interested in an individual plant quarrel relative to the title of property, which, of right, they do not own or control in any way. Usually, the property concerned belongs to the public. As soon as the public assumes control of that which it naturally owns, a great temptation is removed.

4.—The interpretation of grants generally means that the individual has assumed the ownership of public property and has doubts as to his ability to perfect the title. Grants are iniquitous. They mislead the public and the individual. They can develop under any system which is careless. Claimants have demanded as much as 500 cu. ft. of water per sec., and to be delivered through a pipe 1 in. in diameter. Another called for 60 cu. ft. per sec. and proposed to convey it in a ditch 15 in. wide and 10 in. deep. These suggest problems for expert witnesses in litigation which generally leads to prodigal decisions. It is much easier for a Court to prepare a decree, relating to matters foreign to the law, than it is to carry such a decree into effect on the ground. Grants by other, unauthorized, public authorities are generally of equally questionable value.

These grants must all be modified in time. Some day the user will meet a public officer who operates under a law based on wise principles. He will give his testimony and be afforded an opportunity to scrutinize the testimony submitted by his neighbors. A hearing will then be provided for, and at this gathering of all interested claimants, adjustments will be made which will satisfy all, that each is to be protected, that there is to be no favoritism, and that public interests are to be preserved. Finally, an administrative board, respecting all the great and important principles which protect water users and uphold public

interests, will prepare a "decree" defining the various rights in a way that all may understand. Then an engineer of average ability will carry the order into effect. The determination of rights will be made without expense to the water users. Throughout this procedure all claimants will have the same opportunity to be heard and have their interests protected, regardless of financial ability. No litigation will ensue. Water users will be satisfied. Mr. Johnston.

5.—As stated previously, the principle of priority does not bring order when it stands alone. When it is supported by other principles, some of which have been enumerated, "surplus rights" do not exist. Injustice always results where water users are permitted to expand their own uses or to sell "surplus water" to others. In the early days of the West, when the principle of priority stood almost alone, the Courts were inclined to give excessive allotments of water. In one case, a prospective user filed a claim to nearly 1 000 cu. ft. per sec. from a stream that did not carry that much during the summer months. He used only a small part of this volume, but 16 years later, after many other uses had developed, he was permitted to sell what he did not need for his own purposes. The granting of "surplus rights" always results in litigation. It is unfortunate that those injured are generally obliged to resort to the same tribunal for relief as that responsible for the existing condition. It is natural, under these conditions, that relief comes slowly.

6.—Prescription is an outgrowth of the doctrine of riparian rights. This doctrine does not permit of the diversion of water from streams. Where the Courts find that it has been violated for a number of years and money has been spent in construction, the water user is protected under the theory of prescription. Prescription is the reward extended to water users who violate the doctrine of riparian rights. It is natural that it should lead to some trouble. It was born of trouble. It is almost as iniquitous as the doctrine of riparian rights. Regarding abandonment, the public should insist on the beneficial use of water. If use is not made, there should be a way to terminate the right. The period of non-use should be brief. With so many concerned, and because waters are of so much value to individuals and to the public, the latter cannot consent to witness even a brief period of non-use. The reward which the public exacts at present is less than nothing. To demand continued, beneficial use, does not place the public on a par with the private consumer. The Courts are very slow even to call the attention of private users to the fact that they are slothful. "Unlicensed use" is a new complication, and the terms would suggest a suspicion of public interest somewhere.

7.—The transfer of riparian rights, or almost any right or use to another use, is generally fruitful of litigation. That is why the right to use should belong to the use and not to the user, and always be

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limited to the quantity applied beneficially. There has probably been no more frequent or serious injustice done water users than that which has resulted from transfers of this kind. A riparian proprietor generally uses water for domestic purposes. Instead of calling him a riparian proprietor, or a proprietor of water or water rights of any kind or degree, he should be known as a water user, the kind of use should be defined, and a limit placed thereon. He is blessed by Nature because he has a supply of water at hand. Under any competent administration he has nothing to sell in the way of water or water rights. If others buy his land, they probably pay more, slightly more, because of the proximity of water supply. If some one desires to flood his land, or a part of it, in order that a pond may be created, he sells a part or all of his holdings. He does not sell water or water rights, because he may still wish to exercise his right to domestic use, and, if not, that right lapses. It is the only water right he has. The public still owns the water and the channel of the stream. When the land is flooded, no change occurs except that the purchaser of the land takes the place of the original occupant, and there is a water expanse instead of a ground area. The channel of the stream and the water still remain public. Under the doctrine of riparian rights, the stream cannot be dammed or the flow of water interfered with. That doctrine survives, only to be violated.

Change in character of use should always mean forfeiture of rights, except where the use is to be transferred to a preferred use. It is practically a forfeiture in the latter case, also, except where the principle of priority is upheld. Under the latter condition, the priority is carried with the transferred right to use water. The use can always be disposed of, and the public records do not need to be burdened with the names of users. The public records, in so far as the administration of streams is concerned, simply relate to the uses, their priority, their location, and their extent. These elements are not altered by any change in the ownership of property connected with the use.

8.—The natural flow of streams may be affected by pondage, storage, and by actual diversion. All these things are prohibited by a doctrine which gives every owner of lands abutting on the streams the right to demand the waters of that stream to pass his lands undiminished in quantity and undefiled in quality. Yet these things all prevail, on every stream where the doctrine of riparian rights is presumed to be the mainspring of inspiration, in the United States. Pondage, storage, and diversions would not result in litigation, or even serious conflict, under an administration supported by the principles described.

Though public ownership and supervision of water is of great importance to the public, whether represented as a Nation, a State, or a lesser political subdivision, it is of greater value and importance to water users, who have led in the demand for public control of streams and lakes. Where water users oppose legislation of this kind, we must

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come to one of possibly two conclusions: Either the public concerned lacks confidence in itself, or water users are ignorant of what has been accomplished where public ownership and supervision have been tried.

Mr. Decker makes a very interesting presentation of the relative rights of public and private users. Where the public diverts water from a stream to the detriment of other users, some return from the public, in proportion to the damage done, is natural and just. However, in making this transfer from one use to another, it should be borne in mind that as the public use grows, the private use decreases, and that the public is not buying water rights or water, but is paying damages to the private users because of the decreasing efficiency of the plant concerned in the use, due to partial failure of the water supply. The local public is then taking water from a river that belongs and always has belonged to the public generally. This water has merely been used for a brief period by the private parties. They have invested money and built a plant or plants for utilizing the water in some way. It is not a great undertaking to determine what damage is done when the water supply is reduced by a known volume. The city is generally required to pay for water rights, and other intangible things, on a valuation which would never be countenanced were the private party arranging to reimburse the city in a similar transaction. Assume, for instance, that a city is using the entire flow of the stream for power purposes. A new use then develops. This new use, we will assume, is created by the needs of a farmer and his family. The farmer uses a small volume of water daily, and he must divert this above the power plant of the city. A user of this kind is generally protected, provided he can have access to the river. If he is an owner of riparian lands, the Courts uphold him to the extent of his domestic uses, at least. How the Courts protect the use under the common-law doctrine, we need not try to understand. As we assume a larger number of farmers, each making an equal demand on the river, the city must gradually relinquish its use of water for power purposes. Now, the owner of riparian lands, under the common-law rule (so the Courts tell us), can use the water for domestic purposes and refuse to reimburse the city. The principle of priority would have to be recognized, we must assume in such a case, before the city could demand a return for damage done. Usually, we have the domestic uses of a city against the power interests of private parties. The domestic uses of a city support life. If the private user for power purposes has his money returned to him, so that he loses nothing by the investment in power development, society at large receives the highest, general benefit. By assuming that the public, the municipality, is the user of water for power purposes, and that it must give up that use as more important and necessary uses develop, the question may be viewed in a light that is possibly new to many who have studied it from other angles.

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An example from an adjoining field may serve as an illustration. A city is asked to extend a franchise. Representing the public, it is naturally presumed to give the franchise to some company which flatters itself with the announcement that it is to provide a "public utility." The franchise is given, the only condition attached being the construction of certain works, a part of the "public utility." Before the franchise expires, the city decides to purchase the private, corporate property and try to do something for itself. We might assume that the corporation would volunteer to turn over the property at actual cost, less depreciation, plus appreciation, plus renewals and replacements, etc., etc. The city is generally charged a tidy sum for such part of the franchise as has not expired. If the city had considered the value of the right to do business in the beginning, it could, by no possible argument, have been shown that a franchise should ever have been given, because it had the opportunity then to serve itself in perpetuity.

An example within an example may not be out of place. A new road is to be opened. Some private lands must be secured for highway purposes. The public has power to condemn lands for such purposes. The value of the land may be fixed by agreement between public authority and private owner. If a friendly agreement cannot be reached, a commission or board of appraisers is appointed, and a value is established. Of course, the land owner has a right to appeal. He seldom appeals, because he is paid a fair price for the land, and he knows this, if he does not admit it. Now, the land thus taken from the farmer may have produced crops which returned an annual net profit of $15 per acre. The farmer and his heirs had the right to enjoy this profit for an unlimited number of years. Lands that return a net, annual profit of $15 per acre are worth $300 per acre, assuming a rate of 5 per cent. The farmer, under the franchise theory, has the right to demand something because of the right he enjoys to do business for himself. As this right is unlimited as to time, the farmer simply capitalizes his annual, net profit at 5 per cent. This enables him to add another charge of $300 per acre, or $600 in all. We know that the farmer is never permitted to impose on the public in this way, because he is dealing with men, usually his neighbors, who fully understand the value of his land and who also understand the necessities of the public.

After the city has been penalized by the Court for having given the franchise, the corporation adds other charges. One of these has the illusive, yet phonetic title of "going value." It is true that many private water users pay taxes, but they do not pay taxes on the value of water rights, which they frequently claim to have for sale. They do not pay taxes on the value of the unexpired franchises; they do not pay

taxes on going value. These are for sale to the public, not for the public to tax. Mr. Johnston.

Little money is available, usually, to protect the interests of the municipality. The able professional man is tempted to go to the aid of the private party, and he generally does. The sympathy of the Court for the same party may easily be aroused, because the judge, in his professional journey from office boy to the bench, may have lived in an atmosphere hostile to public initiative and unfriendly to the protection of public interests.

In the administration of streams, in the supervision of drainage, in the promotion of public health, and in many similar fields of service, the principles which should control can only be discovered and fully protected by specialists in the various branches. The public rules, which we know as laws, and which should incorporate these principles, can best be framed by these specialists. The specialist does not invade the domain of the attorney in so doing. Laws relating to these special subjects are, under this practice, couched in good, plain English. They are administered by professional men who understand them. Rarely are they misconstrued.

Wise principles, even when they relate to matters of pure law, do not always develop under the exclusive jurisdiction of the Courts. It may be possible to give one reason for this. When some question, which involves principles of importance to many, is brought to the Court, only a few interested in its solution are able to be represented. As time goes on, many of these find themselves unable to defray the expense of extended litigation and hence withdraw, so that when testimony is finally presented and argument made, only those who are financially strong remain. The principles which might have been supported by the many who were unable to enter the contest in the first place, and those which were upheld by those who were obliged to withdraw, are never presented to the Court. The decision finally reached must be based largely on the principles supported by the few. This decision probably places the few, fortunate litigants in a position where they may profit, possibly, at the expense of those who were unable to protect themselves in the Court proceeding. When later litigation is indulged in, these fortunate persons are able to meet their rivals in the Court, fortified with greater financial resources and reinforced by a valuable precedent. As time runs on, under such a system, the Court continues to get farther away from the principles which should govern.

When we try to follow the development of underlying principles, we find ourselves immediately interested in men who have led in the various fields of human endeavor. The university has been slow to take an active part in the formulation and discussion of principles. In a way, however, it has prepared men for that service. Only a few years ago,

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a professional man from the university was either a minister of the gospel, a lawyer, or a doctor of medicine.

As the human race developed, other professions were born, and some of these have grown until they have recognition almost as broad and as general as that accorded the three older branches referred to. Engineering is probably the strongest of the younger professions. It is the province of the engineer to accept the gifts of Nature and to transform them in such a way as to afford the greatest happiness and the highest service to the race he represents. This work is distinct from religion, law, and medicine. In the various branches of engineering activity, principles of great value to the public are discovered by the engineer. These must be observed if the best and highest uses of Nature's gifts are to be protected and enjoyed. The engineer has a great field. He is responsible to the Maker of all things on one side and to the millions of his own kind on the other.

Four boys grow up together. They enjoy equal educational advantages. They come from homes and from environments that are similar. Each selects one of the four great professions, and proceeds to fit himself to be of service to those with whom he must live, and, incidentally, to support himself in a way that will reflect credit on one who spends years of his life in preparation for work. As years pass, these four men assume high places in their various professions. The lawyer is finally elevated to the bench, the minister becomes a high officer of the church, the physician leads his profession in at least one line of work, and the engineer finds new methods of converting the gifts of Nature into blessings to humanity. Up to this time, these men have been practicing their professions along with many associates. Now, they also become authorities as to principles that concern the work of their own professions. Further, each should be in position to define the principles which should establish the relations of individuals and the public with the things that are studied and understood only by men of their own professions.

When some of these authorities assume an interest in public affairs, there are some obstructions in their paths. A concrete example may serve to illustrate. Assume that the time has arrived for a forward step in matters of public health. The physicians, interested in performing a great public service, naturally advise with the leading men of their own profession. They frame a measure, a duty they are eminently qualified to perform. The measure is submitted to the law-making body. Not a single member of that body represents the medical profession. The committees to which it is referred have no special knowledge of the subject to which the bill relates. Physicians who are active, who are willing to give of their own time for the good of the public, and who have no hope of or desire for reward, appear before these committees. These physicians may not be adept in advancing

arguments to those whose training does not lead them to appreciate readily the importance of the pending measure. The committees may finally report favorably. When the measure is placed before the law-makers, the arguments advanced in favor of its passage must come largely from the committees. Its prospects for passage are not bright. If passed, the chances for amendments, which practically nullify the purpose of the bill, are numerous. The bill finally emerges from the legislative test of "running the gauntlet" and becomes a law; but probably is crippled from the beginning. The law probably provides a simple administration. The Governor, or some other officer, far removed from frequent contact with problems of public health, has the appointive power. He may select a competent board of health. He may select an attorney from Podunk, a garbage collector from Utopia, and another politician from Anywhere, as the board to carry this new legislation into effect. Mr. Johnston.

In a short time the entire law may be questioned by some one who claims a "vested" right in maintaining a public nuisance, or in doing things which seriously interfere with public health. Now, this legislation relating to public health, so ably supported by unselfish members of the medical profession in the beginning, is to be taken into the Court, which has had no opportunity to study the principles on which the law is based. It is possible that the great physician who understands the value of the law and who appreciates fully the principles on which it rests, may be called as an expert witness in the initial hearing of the case. Those who have occupied such a place realize how little information such a man is permitted to give, particularly in the definition and support of principles. The witness is confined largely to testimony which might be given by any practitioner. He, better than any man living, is in position to say what principles should govern. An appeal is taken to the Court of last resort, and all testimony is transmitted in documentary form. The great physician is shoved out of the field of service, and the public loses his aid. The great lawyer, the judge in the Court of last resort, never comes into contact with the great physician. He has not followed his work, and he knows as much about the problems of public health as do many others who have not enjoyed close association with them. He cannot appreciate the importance of the principles that the great physician supports, yet he has inherited the right to enunciate the principles that the public is presumed to observe. His decision must be made according to his own light; and there is little new light filtering through to the tribunal of last resort, where problems, so foreign to the law, are considered by the Court. Though we have been laboring for many years under the delusion that a judge, whose training and experience have been confined to the realm of law, may be sufficiently educated, instructed, and informed by testimony relating

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to a subject, probably technical and at least foreign to him, so that he may make a decision which is equitable and just, still we must condemn the practice when we give it serious consideration. An engineer would not assume that it would be possible for lawyers to make plain to him, in a brief and somewhat stormy contest, the value and importance of great, fundamental principles that underlie problems of pure law. In such a case, the attorney would be the witness. He would be confined to the very elementary phases of the principles under consideration, and, following the prevailing practice of our Courts, he would be bulldozed, persecuted, and villified, while he hesitatingly felt his way along, always knowing that he was not presumed to express his views fully and truthfully and in the light of his special knowledge of the subject. The engineer, acting as judge in this imaginary court, would be unappreciative of the important phases of the questions brought to his attention. He would not be prepared to recognize underlying principles of pure law. He would generally be somewhat prejudiced, because some trivial and unimportant matter, relating to his own profession, was indirectly concerned with the really great principles before him. He, therefore, would receive so little help from the witness, and from those conducting the case, that he would have more confidence in his own judgment, usually, than in anything that came to him in the way of testimony and argument.

We can probably ascribe the unfortunate condition of our streams, to-day, to our willingness to permit them to be governed under principles that have been inherited from a decayed civilization or evolved by a profession which is educated, prepared by practice, and finally qualified to perform efficient service in a foreign field.

Finally, a tribute to the great jurists of the past, who studied the best and wisest rules of their own times and then bequeathed to us the benefit of their wisdom and judgment. We have no more valuable inheritance, possibly, than that which has been created by students of the science of law in former years. The principles underlying pure law concern men and their relations with each other. If "the proper study of mankind is man," the study of law is involved therein. Strange as it may seem to some, the legal profession is not so much concerned in general legislation. The Court excuses no man because of ignorance of legislation. There are no professional qualifications prescribed for membership in law-making bodies. Legislation, therefore, is a matter of common concern. It belongs to no particular profession. Speaking generally, other professions than law are supported by principles found in Nature. Man may discover them—he cannot originate them or change them. The principles involved in legislation, which concerns the work of professions of this kind, are all foreign to the profession of law. Injustice is done the Court when it is required to construe such legislation, except where some incidental

feature of pure law is concerned. Greater injustice is done the Court when it is required to undertake the administration of things that are largely controlled by principles which are unknown in the science of law. Therefore, when legislation—embracing principles which concern the relation of men to things and to policies foreign to the science of law—is to be framed, interpreted, administered, or scrutinized, this duty should be performed by men educated and trained in those particular branches, more than in law. Mr. Johnston.

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EXTERNAL CORROSION OF CAST-IRON PIPE

Discussion.*

By Messrs. W. J. E. Binnie and Leonard S. Doten.

Mr. Binnie.

W. J. E. Binnie,† Esq. (by letter).—The author is to be congratulated on his exceedingly interesting and most useful paper. The subject of corrosion of metal pipes is one of great interest to all water engineers, and in the past too little attention has been paid to the study of the character of the various soils in which pipes are to be laid, with the consequence that their life has sometimes been lamentably short.

It is generally considered that, owing to the foundry skin, cast-iron pipes do not corrode so rapidly as steel. It is very seldom, however, that even a cast-iron pipe, when taken up after being several years in the ground, does not show external corrosion, to a certain extent, thus pointing to the fact that the usual coating materials do not afford permanent protection.

Fortunately, the corrosion is generally confined to a certain depth, that is, pitting occurs, accompanied by nodular incrustation, which, at first local, spreads in area as time goes on, so that the entire surface may eventually become corroded; but this pitting, whether in cast-iron or steel pipes, seldom extends to a depth of more than $\frac{1}{8}$ in.; and when the surface is entirely covered with nodular incrustation, a stage is reached when further deterioration of the metal ceases.

With cast-iron water pipes, the factor of safety is large, and the necessary minimum thickness for good castings is considerable, with the consequence that the pipe is still strong enough, even when attacked by ordinary corrosion both within and without, though its carrying capacity may be affected seriously. Special cases occur, however, such as those which the author brings forward, when the material

* Continued from November, 1914, *Proceedings*.

† London, S. W., England.

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of the pipe itself is entirely changed without loss of outward form, and the pipe becomes useless; and papers such as this are of great service in showing the engineer what is to be expected if he lays pipes in similar ground without taking special precautions.

The experience of the writer's firm leads them to believe that the depth of pitting does not differ much, whether the material is cast iron or steel, but the loss of $\frac{1}{4}$ in., due to internal and external corrosion together, will be of very serious consequence in the case of the steel pipes.

The writer's firm was engaged by the Western Australian Government, in conjunction with Sir W. Ramsay, and Mr. Otto Hehner, to report on the remedial measures which should be adopted to arrest the corrosion which was taking place in the pumping main for the supply of water to the Western Australian Goldfields. This main was 350 miles in length and 30 in. in diameter. It was corroded internally, owing to the faulty coating and the character of the water, so that some sections had lost 50% of their carrying capacity; and externally to such an extent that the main was sometimes completely penetrated.

The external corrosion was most active in saline and in clay soils, but where the main was laid through sandy light soil, corrosion was generally slight. The remedial measures which were undertaken to deal with the external corrosion where the pipe had to be underground consisted in opening up the trench, cleaning and scraping the outside of the main, re-coating, and wrapping it with tarred Hessian. In addition to this, it was suggested that the pipe should rest on a bed of lime concrete, and should be surrounded with a layer of slaked lime when the trench was refilled, but it is not known whether or not this was actually done.

The writer felt fairly confident that the tarred Hessian, if perfect, would in itself be a sufficient protection, but it was not at all an easy matter to wrap this Hessian so that it would adhere to the entire surface of a lock-bar jointed steel pipe, such as the Coolgardie main.

To remedy the internal corrosion, the writer suggested the addition of lime to the water, and also the removal of the dissolved air, as experiments had shown how easily this could be done and how slowly corrosion took place with this air-free water. It is believed that the Western Australian Government has carried out the first recommendation, but, as far as known, it has not tried the latter.

The writer's firm is now laying a steel pipe line from Wales across the estuary of the River Dee to Birkenhead, in Cheshire. The pipe, where it crosses the estuary, is in salt marshes slightly below high-tide level for several miles, and owing to the high pressure and other reasons it was desirable to use steel pipes. These are flanged pipes,

and, owing to the restrictions placed on the Birkenhead Corporation, had to be laid underground. Mr. Binnie

The writer's firm has decided to take the following precautions:

(1) To wrap the pipes externally with two layers of Hessian treated with a bituminous solution similar in composition to that used for cables.

(2) To lay the pipe in lime concrete except at the flanges, which will be surrounded with puddled clay mixed with slaked lime in the proportion of 6 parts clay to 1 part slaked lime, in order to facilitate the replacement of a faulty pipe.

LEONARD S. DOTEN,* M. AM. SOC. C. E. (by letter).—The writer has had considerable experience in the use of cast iron in sea water, chiefly in the form of water mains and bearing piles for wharves. As a result of this experience, he concludes that cast iron is much superior to steel or wrought iron in resistance to corrosion. Mr. Doten.

The author states:

"Five separate theories have been advanced to account for the rusting of iron, but, at present, three of these have been practically eliminated, leaving only two over which the fight still wages: the acid theory and the electrolytic theory."

In the majority of cases corrosion can be explained most satisfactorily by the latter theory, but no one theory accounts for it under all conditions; corrosion may be either the result of chemical action or electro-chemical action, depending on controlling conditions. The cases of rapid disintegration of water pipes cited by the author were undoubtedly examples of electro-chemical action. The writer does not mean to imply that the electrolytic theory, as usually understood, accounts for the corrosion satisfactorily in those cases.

On page 1647† the author states that "the salt of the ocean is particularly destructive to cast iron, * * *." The writer believes this to be true only under certain unusual conditions, which will be described later. It is a well-known fact that the superstructures of many of the wharves along the Atlantic Coast are supported by cast-iron piles. Cast iron is the most durable material, available for use, for wharf piling in salt or fresh water. The Government pier at Old Point Comfort, Va., is a good example of this type of construction. This wharf was built about 32 years ago. The writer knows from personal inspection that the piles of this pier are now in practically as good condition as on the date of construction. The steel I-beams resting on these piles have been much weakened by corrosion, scale having formed on them ½ in. or more in thickness.

In 1903, the writer had a very interesting experience, in investigating the cause of the rapid disintegration of a submarine water

* Washington, D. C.

† *Proceedings*, Am. Soc. C. E., for August, 1914.

Mr. Doren.

main and in devising a means to prevent further trouble from this source. The pipe line in question was in Boston Harbor, and connected one of the distributing mains of the Boston Water-Works with the distributing system of one of the forts. The line was approximately 2 000 ft. long. It was of heavy, flexible-joint, coated, cast-iron pipes, 6 in. in diameter, and was laid in a trench 5 ft. deep, and covered with clay, in accordance with the standard practice in that locality. Near the shore of the reservation, a break had occurred which was the third at this point, due to the same cause, since the construction of the line, about 7 years previously.

When removed, it was found that there were several holes in the bottom and sides of the pipe, ranging in diameter from ½ in. to 1 in. Aside from these holes, the general appearance of the pipe was good, and the coating apparently was intact. By very close examination, however, it was possible to discover many small circular areas where the coating was elevated slightly above the general surface. At these points, it was possible to cut deep holes with a knife, in some cases extending completely through the shell of the pipe. The appearance of the disintegrated metal was similar to that of graphite, which is characteristic of cast iron which has undergone the change produced by electrolysis. On exposure to the air for a time, the substance became harder.

On account of the close resemblance of the corroded pipe to that which had been injured by electrolysis, and the fact that the line affected was at a place where injury due to this cause would be most likely, a test was made to ascertain whether an electric current was passing through the line, but gave a negative result.

Following this, another theory to account for the corrosion came to the writer's mind. It had been noted that there was a large accumulation of finely divided anthracite coal on the beach in the immediate vicinity of this line. This coal had been brought there by the action of the tidal currents from the opposite shore of the island, where a coal barge had been wrecked several years before, and had come into contact with the pipe in the trench. Under the pipe there were large masses of cemented gravel having the color of iron rust.

Furthermore, each time a defective pipe had been replaced with a new one, it had been necessary to extend the trench a little farther, in order to make a suitable connection with the sound pipe. The small pieces of coal became distributed over the bottom of the trench to a greater or less extent during this operation, and, as a consequence, more pipes were affected. This probably accounts for the fact that only two lengths were removed the first time, whereas about eight lengths were removed the last time.

Taking these facts into account, the writer devised the theory that the action causing disintegration of the metal was electro-chemical,

and resulted from the contact of the coal (carbon) with the iron in an electrolyte (sea water). An experiment was made to ascertain the correctness of the theory. A crude electrolytic cell was formed by using a large glass jar of sea water, a piece of the pipe metal, and a large piece of anthracite coal. Iron wires were used to connect the two poles. Connecting these wires to a galvanometer the writer obtained an initial deflection of 5° and a constant deflection of 3½° when the instrument was adjusted for the proper external resistance. Gas bubbles formed on the poles, in the manner characteristic of electrolytic action. Mr. Doten.

As a result of this experiment, the writer was convinced that the theory was correct and that the metal of the pipes had been carried away by electrolytic action resulting from local currents.

The report which was made to the War Department, dated February, 1904, contained the statement that, in the writer's opinion, the disintegration of the pipe was due to galvanic action between the coal and iron, and a recommendation that the new pipe as soon as placed be encased with a rich mixture of concrete. The work was carried out in accordance with the recommendation of that report. No further trouble due to this cause has been experienced since that date.

As electrolytic action is produced when carbon is placed in contact with such metals as zinc, tin, and iron in an electrolyte, it is the writer's opinion that all forms of coal, peat, and other vegetable material which has been more or less carbonized, as well as many other substances, also produce electro-chemical action under similar conditions. The rapid corrosion of the water pipe lines at Elizabeth, Perth Amboy, and Atlantic City, N. J., Syracuse, N. Y., and those lines laid in cinders from power plants, could be accounted for most satisfactorily in this way.

Undoubtedly, cast-iron wharf piling and the steel or iron supports for structures in sea water have been weakened by such action. Therefore, the writer believes it would be unwise to use cast iron or steel in the substructures of coal wharves in such locations. Electrolytic action would be likely to take place at the bottom.

The author's statement that "to get the full corrosive effect of sea water, plenty of air is essential" does not well apply to cast-iron piles, as many of those which have been in place 30 years or more show little evidence of corrosion. It applies more accurately to wrought iron and steel. In the case cited by the author, the piles failed at the bottom, where the air was probably least plentiful.

It is also stated that the meadow soil at Atlantic City was free from organic acids. The soil conditions were suitable, and there was sufficient sodium chloride present to make electrolytic action possible. The more rapid corrosion of this pipe at the top than at the bottom could be accounted for by the accelerating effect of the sun heat of summer or by a difference in the character of the soil.

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A METHOD OF DETERMINING STORM-WATER RUN-OFF

Discussion.*

By W. E. Fuller, M. Am. Soc. C. E.

Mr. Fuller

W. E. Fuller,† M. Am. Soc. C. E. (by letter).—The author states:

"There is need for a formula of general application which can be used in a new locality with some reasonable assurance that it will be fairly appropriate, will be applicable for large areas or small, flat or steep slopes, and heavy or light rainfalls.

* * * * * * * * * *

"The writer has developed such an empirical formula for storm-water run-off, and it is presented herein, together with a detailed comparison with all published gaugings available to him."

In order to cover such varied conditions as these, it is essential that the relations existing between the different factors in the formula shall be consistent. The author does not state that there is any limit to the size of the catchment areas for which his formula is applicable. His diagrams give values for his factors which would be proper for large catchment areas, and he makes comparisons of streams having catchment areas up to 100 sq. miles. It may be assumed, therefore, that he expects his formula to be applicable for such areas. The writer has studied the formula, particularly as to its application to large areas, and is convinced that the relations proposed are not true for such cases.

In order to test it, the writer has made computations by using the author's formula, the results of which are shown in Fig. 8. This is a plotting of the effect of different factors on the run-off for different catchment areas. In making these comparisons, values for the different factors were assumed, giving a somewhat wide range in order to

* Continued from December, 1914, *Proceedings*.

† New York City.

Mr. Fuller.

cover the ordinary conditions which will be met. The relative effect of the factors is shown by the change in q due to changing one factor while all the other factors are kept constant, or, in the case of L and A, in proportion. For all plottings showing changes in C, S, and M, K is taken as 1, and L is taken as the square root of A (A being in square feet).

In Table 23 are given the percentages of change in q due to changing the different factors for 10, 1 000, and 100 000 acres, as taken from Fig. 8.

TABLE 23.—RELATIVE INFLUENCE OF DIFFERENT FACTORS IN MR. BUERGER'S FORMULA:

$$q + \frac{L}{79\ \sqrt[5]{C\,A\,S^2}\left(24 + \frac{16}{\sqrt{m}}\right)} \times q = \frac{3.33\ \sqrt{m}\ K\,C}{\sqrt{m} + 0.87}.$$

Area, in acres.	PERCENTAGE OF CHANGE IN VALUES OF q DUE TO GIVEN CHANGES IN THE VARIOUS FACTORS.				
	L, in feet, changed from $2\sqrt{A}$ to $\sqrt{A}$.	S, changed from 5 to 50.	m, changed from 5 to 50.	K, changed from 1 to 2.	C, changed from 0.15 to 0.8.
10	8– 15	9– 12	26–29	100–115	440–490
1 000	27– 48	35– 39	19–24	105–120	500–600
100 000	70–110	100–110	12–16	115–135	630–850

It will be seen from Fig. 8 and Table 23 that the relative effect of all the factors varies for different values of A.

The effects of the factors, as shown in Table 23, are not for the extreme cases. This may be seen by an examination of Plate XIV. L and S do not affect the factor, P, occurring only in the factor, N. N is directly proportional to L, and is inversely proportional to $S^{\frac{2}{5}}$. Reducing L by one-half then reduces N by one-half. For small values of N, this affects q by only a slight percentage; but, for the largest values of N given in the diagram, it increases q by from 2.3 to 2.5 times. Changing S from 5 to 50 reduces N by about 2.5 times. Such a change in N, though changing q by only a slight percentage for small values of N, increases q more than 3 times for the largest values of N. K does not occur in N, but P is directly proportional to it. Doubling K then doubles P. For small values of N, this practically doubles q, and, for large values of N, q becomes some 2.35 times as large.

Effect of Changing K.—The author states that "a selection of the frequency with which the drains are to run full will determine the

Mr. Fuller.

INFLUENCE OF VARIOUS FACTORS IN THE BUERGER FORMULA

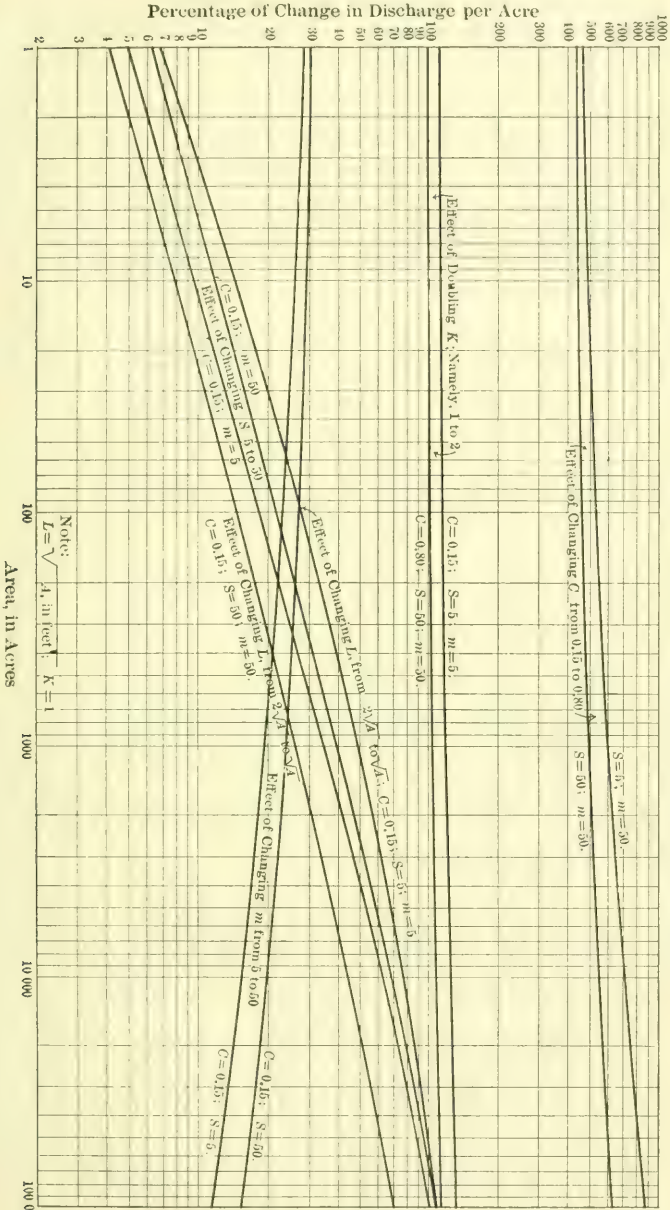
Percentage of Change in Discharge per Acre
Area, in Acres
Effect of Doubling K, Namely, 1 to 2
Effect of Changing C from 0.15 to 0.80
Effect of Changing L from √A to 2√A
Effect of Changing m from 5 to 50
Effect of Changing S from 5 to 50
C = 0.15; S = 5; m = 5.
C = 0.80; S = 50; m = 50.
S = 5; m = 50.
S = 50; m = 50.
C = 0.15; m = 50
C = 0.15; m = 5
C = 0.15; S = 50; m = 50
C = 0.15; S = 5; m = 50
C = 0.15; S = 50
C = 0.15; S = 5
Note:
L = √A, in feet; K = 1

FIG. 8.

Mr. Fuller.

value of K to be used". If this is the case, the effect of changing K should be relatively the same for all areas, but, with the formula, doubling K affects large areas considerably more than it does small ones.

Effect of Changing L.—L, which is the length of the longest sewer, has a very much greater effect on large than on small areas. That this effect in large areas is too great may be shown by the simple comparisons in Fig. 9, which shows two catchment areas, (a) and (b), alike as to values of S, m, C, and K, one of which is twice the size

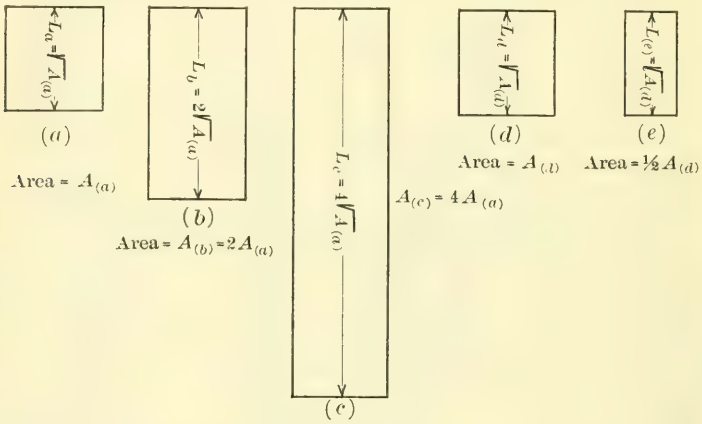

FIG. 9.

of the other. It is assumed that the longest sewer is equal to the longest dimension of the area, or, in case of Area (a) = $\sqrt{A_{(a)}}$ and in Area (b) = $2\sqrt{A_{(a)}}$. Assuming that values of S, m, C, and K are such that $N_{(a)} = 5$, and $P_{(a)} = 1$, the value of $N_{(b)}$ may be found by substituting in the equation,

$$N = \frac{L}{79\sqrt[5]{C\,A\,S^2}\left(24 + \frac{16}{\sqrt{m}}\right)}$$

the values of $L_{(b)} = 2\sqrt{A_{(a)}}$ and $A_{(b)} = 2A_{(a)}$. Then

$$N_{(b)} = \frac{2\sqrt{A_{(a)}}}{79\sqrt[5]{C\,2A_{(a)}\,S^2}\left(24 + \frac{16}{\sqrt{m}}\right)},$$

or

$$\frac{2}{\sqrt[5]{2}}\,\frac{\sqrt{A_{(a)}}}{79\sqrt[5]{C\,A_{(a)}\,S^2}\left(24 + \frac{16}{\sqrt{m}}\right)} = \frac{2}{\sqrt[5]{2}}N_{(a)}$$

$$= 1.74 \times N_{(a)} = 8.7.$$

$P_{(b)}$ will equal $P_{(a)}$ or 1. From Plate XIV it is found that $q_{(a)} = 0.117$ and $q_{(b)} = 0.062$. Mr. Fuller.

The total run-off from the areas will then be $Q_{(a)} = 0.117\ A_{(a)}$ and $Q_{(b)} = 0.124\ A_{(a)}$. By the author's formula, therefore, the addition of another area exactly like the first increases the total run-off by only 6 per cent.

It may be shown, in the same way, that, if three areas are added, as shown in Fig. $9_{(c)}$ the total run-off increases only by about 11 per cent. It can also be shown, by a similar computation, that the square area, Fig. $9_{(d)}$, will, by the author's formula, give 15% more run-off per acre than the area, Fig. $9_{(e)}$, although all parts of the area Fig. $9_{(e)}$ will be closer to the main sewer and the area smaller. These results are clearly not in accord with the true conditions which govern run-off, and the relation of L is not the true one. These obviously inconsistent results can be obtained only with large catchment areas, and prove only that the relation of L is not correct for such.

Effect of Changing the Factors, S and m.—These factors represent the average slopes of the drain and the ground, respectively. Fig. 8 and Table 23 show that the relative effect of increasing S is much greater for large than for small areas, and that the reverse is the case for m. This does not seem to be consistent. In the writer's opinion, the effect of S on large areas is entirely too great.

Effect of Changing C.—C ("the ratio of rainfall eventually reaching the sewers to the total rainfall") has a much greater effect on large than on small areas. In the extreme case which came within the writer's computations, increasing the value of C by 430% increased the value of q by 850 per cent. With larger areas and other assumptions, higher values in the percentage of change could be obtained. That is, the rate of run-off in the sewer may be increased twice as much as the increase of run-off from the area. To the writer this seems to be inconsistent.

The use of average slopes can only satisfy average conditions. The relation between the slopes for different parts of the basin is the real factor. Steep slopes on the drain or on the ground near the outlet may tend toward smaller run-off, rather than larger, as the bulk of the water from the near-by areas runs off before the peak of the discharge caused by the water from more remote areas is reached. Steep slopes on the remote areas will, in general, increase the run-off. For the conditions of maximum run-off, the slopes for all parts of the area must be so related as to bring as large a concentration as possible.

The effect of the slope is also dependent on other factors, such as the shape of the basin and the coefficient of run-off. Needless to

Mr. Fuller.

say, the problem of the effect of the slope is an extremely complex one. The rational method for obtaining the run-off by considering the different sections of the area separately, takes into account the relative slopes, and, therefore, is to be preferred to any formula based on average slope.

The author suggests that the selection of K will give the frequency with which run-off will come. The rainfall is only one of the factors involved. The conditions existing at the time the storm comes are of much importance. In a suburban or undeveloped district, the condition of the ground, whether saturated or frozen, the quantity of snow on the ground, and other factors, have a controlling effect on the run-off. The frequency is a function of the coincidental occurrence of conditions involving many factors. This frequency may or may not follow the same law as the frequency of occurrence of rainfall intensity.

The writer desires it to be understood that his criticism of the paper is limited mainly to the use of the formula for determining run-off from areas for which values of N are large. This will occur, in general, when A is large. For a more limited use, perhaps covering all but the largest of sewer areas, the formula may give results approximately correct for average conditions.

The relations existing between the factors, however, are not the true ones, and for determining run-off from very large sewer areas and from streams, the formulas, in the writer's judgment, should not be used.

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THE CLARIFICATION OF SEWAGE BY FINE SCREENS

Discussion.*

By John H. Gregory, M. Am. Soc. C. E.

Mr. Gregory.

John H. Gregory,† M. Am. Soc. C. E. (by letter).—The author has presented a paper which cannot fail to be of great interest to all who in any way have to do with the screening of sewage, and he is to be congratulated on having been able to secure so much detailed information on foreign works. That such information can be secured, the writer knows from experience, but this is not always as easy as the reader may imagine. To the writer, the paper is especially welcome, for it brings back pleasant recollections of many months spent on the Continent and in England examining similar works. It was his good fortune to be abroad in 1909, and again during the summer of 1914, and although he has not visited all the works mentioned by the author, he is familiar with many of them, and many have been examined twice.

That fine screens have a definite field in sewage disposal cannot be gainsaid, but the writer is not so optimistic as the author as to the results accomplished. This same point has already been brought out by others who have discussed the paper.

The paper brings up so many points that a discussion of all is hardly feasible, but there are a few matters which may be mentioned. Fine screens have come to stay, but, before trying to draw too definite conclusions as to their efficiency, much more information and detailed data are needed. All sewerage engineers recognize the difficulty of securing representative samples of sewage as it arrives at

* Continued from January, 1915, *Proceedings*.

† New York City.

Mr. Gregory.

or passes through works and, in judging of the work accomplished by screens, this must at all times be clearly borne in mind.

Many of the author's data are from European works, and in studying the results obtained it must be remembered that there is a great difference between most European sewages and those generally met with in the United States. In Europe, the sewages are, in general, very much stronger than those in America, perhaps, on an average, three times as strong, and, in the writer's opinion, it does not necessarily follow that as good results can be obtained in screening the weaker American sewages as seem to have been obtained with the stronger sewages abroad.

In comparing the efficiency of fine screens and settling tanks, the writer would place a higher relative efficiency on properly designed and operated settling tanks than the author appears to have done.

There is one feature in connection with the operation of screens which the writer feels should receive much more consideration than the author seems to have given it, that is, the handling and removal of the screenings. Screenings rapidly become offensive and, unless removed quickly, may become a source of great offense at the works. This has been forcibly brought home to the writer when examining some of the works cited by the author. The handling and removal of screenings from a screening plant is just as important as the handling and removal of sludge from settling tanks, and provision must be made for doing this promptly if offensive conditions are to be prevented. In many works which the writer has examined the question of the handling and removal of the screenings seems to have been of secondary consideration.

It has been suggested in one of the discussions that, in the treatment of packing-house wastes, by the use of fine screens in connection with settling tanks, a saving in the sludge storage capacity of the settling tanks and in the area of sludge drying beds could be secured, and that thereby a substantial saving in the cost of tanks and drying beds should result. Under some conditions this might be the case, but the saving may be more apparent than real. The saving in cost of tanks and drying beds, in part, at least, would be offset by the additional cost of the fine screens, and at first sight it would not appear reasonable to reduce the size of the settling chamber, as the removal of a portion of the suspended matter would not affect the volume of the sewage appreciably. In fact, the addition of the fine screen may increase instead of decrease the cost of the works.

Again, cost of construction is not the only factor to be considered. There is the cost of operation of the fine screen and the removal of the screenings which must be taken into account, to say nothing of the fact that screenings removed from packing-house

wastes are perhaps of the most offensive kind, and if removed from the sewage, the likelihood of nuisance would be greatly increased. Mr. Gregory.

As stated previously, screenings quickly become extremely offensive and, from the writer's observations, the surest way to prevent nuisance from such materials is to keep them away from the atmosphere, and the simplest way to do this would appear to be to keep them in the sewage and handle them in the settling tanks. Minimum cost is not the only desideratum in building a sewage disposal works, absence from offense is generally equally important.

The use of the fine screen in connection with settling tanks to prevent the formation of scum is another matter, and it may be that here the fine screen could be used to advantage, but it seems to the writer that the data are yet too meager to enable one to draw definite conclusions on this point.

The writer feels personally indebted to the author for having presented such an interesting paper and for having brought together information and data which have been so scattered.

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT.

Discussion.*

By Allen Hazen, M. Am. Soc. C. E.

Mr. Hazen.

Allen Hazen,† M. Am. Soc. C. E. (by letter).—The water supply conditions in San Francisco are unique in some respects. Notwithstanding the exhaustive treatment of the author, some of the peculiarities of the situation do not seem to be adequately reflected in his paper. The comparison between San Francisco and Los Angeles is a natural one; it is one that has been made very frequently, and will continue to be made, although there are fundamental differences in the water supply conditions of the two cities.

Los Angeles was located on a water supply. The primary reason for its existence was that water was readily obtained from the Los Angeles River. This excellent supply, carefully developed, has served the population of the Los Angeles district up to the present time, and it is capable of a certain further development. This water is close at hand and at a relatively high elevation, so that it can be, and is, delivered in the city at a very low unit cost. Under these conditions, low water rates have been possible and are natural.

The people of Los Angeles, realizing the limit of the Los Angeles River supply and the effect of that limit on the ultimate growth of the city, went about securing a new supply from the Sierras. The undertaking has been carried out during the last 10 years in a way that commands the admiration of the world. In building these works, the city was extraordinarily fortunate in finding a route for an aqueduct on a continuous down grade from the source in the

* Continued from January, 1915, *Proceedings*.

† New York City.

Mr. Hazen.

mountains to the city. Both the source and the intervening land were at sufficient elevations to allow a gravity supply. Indeed, the elevations were more than sufficient, and a large surplus head is available for use in developing electric power. Under these conditions, notwithstanding the long distance, the cost of introducing Sierra water has been surprisingly low.

San Francisco is located on a harbor. The harbor is a magnificent one, and justifies the location. The site of the city was no doubt selected without regard to water supply, and local sources have proved sufficient for many years, although, as the years have gone by, it has been necessary to go farther and farther afield and to build new works at increasing cost. As time goes on it will be necessary to go still farther, and finally to introduce water from sources entirely outside the district.

A Sierra supply can be brought to San Francisco, but, unlike Los Angeles, an economical aqueduct location from the Sierras is not possible. A mountain range must be crossed, and this lies between two wide valleys but little above sea level. Under these conditions, although the distance is less, the cost of a suitable aqueduct will be increased many fold.

Sources now in use are sufficient to supply San Francisco and the whole surrounding district for some years, although additional storage, aqueducts, and other works are urgently needed to make these sources fully available for present and future needs. By going farther there are many possibilities of additional supply, and ultimately the quantity of water that can be made available, if needed for San Francisco, is enormously greater than the quantity that can be made available for Los Angeles; but the cost to San Francisco of the great supplies of the future will be far greater than that of the supplies which Los Angeles has utilized.

The cost of water in San Francisco and in the San Francisco district will never be as low as it is in Los Angeles. The reason is found in the great natural difficulties, and in the cost of the works required to gather, transport, and distribute water in the San Francisco district.

The people of San Francisco may regret that they cannot have water at rates that are current in Los Angeles, but they must remember that the higher rates are part of the price that they must pay for their location on their unique harbor. The harbor is worth the price. If water supply instead of harbor facilities had been the dominating influence in locating San Francisco, it would have been built at Sacramento. There water of excellent quality could have been supplied at a price, except for the cost of pumping, as low as water is supplied in Los Angeles or in any city in the country.

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THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA.

Discussion.*

By R. D. Coombs, M. Am. Soc. C. E.

Mr. Coombs.

R. D. Coombs,† M. Am. Soc. C. E. (by letter).—In reply to Mr. Mitchell's comments‡ on the writer's previous discussion, it would seem to be in order to call attention to a somewhat popular fallacy, that is, that the design and construction of transmission lines is primarily electrical, rather than structural. It is true that the insulation and the electrical capacity of the conductors are purely electrical matters, but it would appear to the writer that all the rest of the work belongs to the department of civil engineering, and that it would have been in order for the authors to have included a complete description of these features in a paper before a civil engineering society.

The foregoing remarks are made in no spirit of criticism, but rather as a request that such matters should more frequently be included in papers before this Society. The writer has no knowledge of the details of the construction in question, other than those furnished by the paper and Mr. Mitchell's discussion, and although from the data at hand it would appear that the line was carefully considered and well designed, the information given is subject to misinterpretation because of its lack of elaboration.

The use of several routes, instead of combining all the circuits on a single right of way, is excellent practice, and might well be followed in other instances.

* Continued from January, 1915, *Proceedings*.
† New York City.
‡ *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Coombs.

Mr. Mitchell has apparently misunderstood the writer's reference to the use of long spans in rough country. The suggestion was not that an "average" span of 700 ft. was not used with the Type A towers, but that if the "standard" span on a Type A tower were 700 ft., it would be improbable that the average span with those towers in rough country would be 700 ft. In other words, there was a difference in height of only 3 ft. between the Type A and Type C towers, and it was apparently stated that the standard span of the former was 700 ft., and of the latter 600 ft. There is quite a difference practically between the meaning of the words "average span" and "standard span", the former being the final result, and the latter the original design, which provides a theoretical clearance above the ground in flat country. Therefore, unless the towers can occupy hill tops, or an extra clearance is allowed in the design, it frequently happens that intervening elevations, or the loss of clearance on hillsides, materially decrease the actual span length.

In view of the fact that a careful investigation was made of the climatic conditions, the writer's previous suggestion, advocating a heavier sleet load, is perhaps not necessary. It will be found, however, that sleet may be encountered, during the probable life of a well-built transmission line, in a great many sections throughout America, even in the South. There may be some question, moreover, as to the desirability of providing a sag corresponding to the standard sleet load, even in non-sleet regions, because such sags decrease both the normal and the maximum loads on the line, and, in general, produce a line which is well able to distribute and equalize excess stresses.

Referring to the additional information furnished by Mr. Mitchell in regard to the wooden pole distribution lines: The use of properly creosoted timber, of course, vitiates somewhat the writer's previous criticism; but he does not consider the use of flexible towers (poles?) as being equivalent to the semi-flexible steel pole suggested.

As previously stated by the writer, "a photograph is an uncertain basis for measurement", and he still believes that he was correct in saying that the appearance of the wood pole shown in Fig. 50 would not indicate its true dimensions. The vertical and horizontal spacings appear to be equal, and, if it followed the usual dimensions of a 22 000-volt line, it would not provide nearly as great clearances as stated by Mr. Mitchell, which, of course, are entirely adequate.

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FUNDAMENTAL PRINCIPLES OF PUBLIC UTILITY VALUATION.

Discussion.*

By Messrs. W. Kiersted and Clinton S. Burns.

Mr. Kiersted.

W. Kiersted,† M. Am. Soc. C. E. (by letter).—The first paragraph of this paper presents the key-note of the author's argument, namely that "the fundamental principles involved in valuation work * * * are to be found only in the law and the interpretation of the law by the Courts." This view as expressed seems somewhat restrictive, for, on reflection, it must be conceded that the fundamental principles of valuation work and all kindred processes are the principles of equity and justice, or those principles which inspired the minds of the framers of constitutional government, which should be the mainspring of all law, and are the beacon lights guiding the Courts in the interpretation of law. Moreover, the author will agree that the interpretation of a law may be modified in the light of new and better evidence and to conform to legislative amendments.

A review of the Court's decisions, from which the author has quoted so freely, must impress one with the design of the Court to place few restrictions on efforts put forth in valuation work, so long as such efforts are inspired and controlled by motives of equity and justice. This is particularly true of the late Mr. Justice Brewer's decision in the case of the National Water Works Company *vs.* Kansas City, Missouri, 62 Fed., 853, with which all engaged in valuation work are familiar, with the exception, perhaps, of that portion of the decision not often quoted and reading as follows:

"The city by this purchase steps into possession of a water works plant, not merely a completed system for bringing water to the city and distributing it through pipes placed in the streets, but a system

* Continued from January, 1915, *Proceedings*.

† Kansas City, Mo.

Mr. Kiersted.

already earning a large income by virtue of having secured connections between the pipes in the streets and a multitude of private buildings. It steps into possession of a property which not only has the ability to earn but is in fact earning. It should pay therefor not the value of a system which might be made to earn but that of a system which does earn.

"Our effort has been to deduce from the testimony that which in this view of the situation can be safely adjudged (the fair and equitable value). The original cost of the works is not accurately and satisfactorily shown. If it would have assisted us in reaching a conclusion, if in consequence of our ignorance thereof we have not placed the value upon the property which it deserves, the company alone is to blame, for by the production of its books it could have clearly shown the actual cost of every part and of the whole of the property. There is a large amount of testimony as to the probable cost of reproducing the system, to which strenuous objection is made on the ground of an alleged temporary and extreme depression in the cost of labor and material.

"We have before us the estimate placed before us by two gentlemen of experience and capacity, appointed as commissioners, with direction to report 'the fair and equitable value', but neither by order of the court appointing them nor by their report are we advised as to what they considered the criterion of the present 'fair and equitable value.' If they added anything beyond what in their judgment was the reasonable cost of reproduction we are not advised as to how much they added or what they took into consideration in making such addition. We have the fact of liens placed upon the property to the extent of three million dollars with the qualified approval of the city officials.

"We have also the statement of the earnings and the estimate of the value upon the basis of a capitalization of those earnings amounting, as stated, at 6 per cent., to four and one-half millions. Rejecting the latter as too high and the cost of reproduction as too low, and taking into consideration the entire history of the transactions between the company and the city from its commencement to the present time, we have sought to place the value upon the property as it stands, with all the connections already made between the pipes and the private and public buildings and with the work which it is in fact doing of supplying all of these buildings with water and receiving pay therefor, and that valuation, with much discussion, comparison of figures and readjustments, we have agreed is three million dollars."

After considering collectively the various Court decisions of rate cases, the question arises, is there not a tendency among engineers, in their work of valuing public utility property, to particularize and to restrict the application of the language of the Court to some theory or particular method of valuation, with the ultimate aim of formulating a rule of procedure to be followed in all such work to the discouragement of those who would follow some other method? There seems to be a tendency in this direction with regard to the repro-

duction method of computing cost of physical property, on the ground that this method, having received the approval of the United States Supreme Court, should therefore become an exclusive one. Mr. Kiersted.

It does not appear, from the opinion of the late Mr. Justice Brewer, that the theory in itself is a complete measure of either cost or value, at least to the extent and in the manner used in the Kansas City case. Other decisions which seem to place considerable emphasis on the "reproduction method" doubtless regard the application of the method in a much broader sense than do some persons actually engaged in valuation work. The author states that reproduction "has not been originated by engineers to torment the Courts, but it has been concluded by the reasoning of the Courts to be a logical and important line of evidence", and then goes on to elaborate certain "conceptual" processes regarded as necessary to a proper frame of mind preliminary to the work of valuation, and to give several illustrations, with the evident intent of broadening and extending the viewpoint of the appraiser, and finally to sum up the application of reproduction to valuation work in a manner more restricted than is to be expected in a paper dealing with the underlying principles of public utility valuation.

The reproduction theory really embraces three distinct methods of valuation:

1.—Present-Day Conditions and Prices.—This is the author's method, and consists of estimating the cost of reproducing the inventoried property as a whole under present-day physical conditions at present-day prices of labor and material, or at prices of labor and material representing the average of a few preceding years. This method allows the unearned increment to be included in cost to whatever extent the prices of labor and material exceed those of the past, and the construction costs, which would be incurred under existing physical and urban conditions, exceed those actually incurred under the original physical conditions. It also depreciates indirectly the actual investment in present-day property to whatever extent the present-day prices of labor and material are below those of the past, and the present-day conditions are more favorable for economical construction than those of the past. Structural costs thus obtained are reduced by the amount of depreciation that may have accrued, on the theory that the property possesses limited life and that any method which admits elements enhancing the value of the property should consistently make allowance for those other elements serving to depreciate the value thereof.

2.—Original Conditions and Present-Day Prices.—This method, fully presented in the Report of the Special Committee on Valuation of Public Utilities, of this Society, consists of estimating the cost of reproducing the inventoried property as a whole, under the original

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conditions, but at present-day prices of material and labor, thereby excluding certain unearned increments, allowed in the previous case, due to change from original to present-day conditions, but including such enhancement as may result from an increase of prices of material and labor, and as may result from a change from a new-structural condition to a permanent, settled, operating condition. Cost of reproduction-new thus obtained is reduced by the amount of accrued depreciation in order to arrive at present value for rate-making after having given due consideration to development costs and similar elements of value.

3.—Original Conditions at Original Prices.—This method consists of estimating the cost of reproducing an inventoried property in a piecemeal manner corresponding with the piecemeal construction of the past, under the original physical and urban conditions at the original prices of material and labor, as nearly as they can be ascertained; and thereby includes no unearned increments. Development cost is computed by the deficit method. Discount on bonds to a reasonable extent is taken as an element of cost, unless provision is otherwise made to amortize the discount during the life of the bond as a charge to operating expense. Exception in the case of urban real estate is sometimes made in giving it a present-day value, on the theory that urban real estate possesses tangible and usable value wholly independent of the public utility with which it is associated. This method differs essentially from the investment method, because, in taking account only of the property inventoried as useful for public utility purposes, it excludes from consideration any items of property which may have been abandoned or passed out of use for one reason or another, thereby depreciating correspondingly the investment. It anticipates, further, that in the future any item of property replaced by a new one is charged off the capital account, and the value of the substitute item is charged thereto. No deduction is made from reproduction as thus determined for accrued depreciation in a rate case, for the reason that the owner of the property should earn on a 100% value of property thus valued, and that, so long as replacements are timely, the property possesses perpetual, or, more properly speaking, unlimited life.

Most of the valuations of physical properties which have been made in the past are a sort of conglomerate, partaking to a greater or less degree of all three of the methods just outlined. Some recent appraisers have attempted to particularize, without any wide difference in the final result. Considered in this light, it can scarcely be conceived that any one of the three methods, logically, fairly, and consistently followed, would fail of due recognition from the Court, although a substantial agreement of the final result of the three methods, respectively, used independently, might be considered more

conclusive evidence of cost or value as the case may be. Moreover, the inventory of a public utility property should be prepared and itemized so that appraisers can use any and all these three methods of valuation if sufficiently reliable data are available for the purpose, as is the case of most of the small non-competitive utilities. With most railroad properties and other large, old, and complex properties the case may be quite different. Mr. Kiersted.

The owners of public utilities would do well to see that the inventory of property is prepared so as to facilitate independent valuation from several points of view, and, in addition, to compile a statement of original costs year by year, as complete as possible, as an aid to the public service commission. For such a commission, in arriving at a value which will be fair and equitable to the rate-payer as well as the owner of the utility, is not likely to be influenced strongly by the view of any single expert or any commission of experts reaching a conclusion by a particular and exclusive method.

Cost to Value.—The author has laid considerable stress on the proposition that cost is not value, and that, after ascertaining the cost, and not until then, should the mind of the appraiser be directed to questions relating to value. It is naturally expected that the Courts, taking a broad and comprehensive view of the question of valuation of public utilities, would discuss the relation between cost and value academically; but those engaged in the practical work of valuation should be able to supply in a specific case the connecting link in the line of reasoning between cost and value when making the distinction. It is conceivable that such a distinction may exist, but it is probable that it more strictly applies to competitive public utilities like railroads than to monopolistic utilities like water-works. The writer in the discussion of Mr. Nicholas S. Hill's paper on "Valuation of Public Utilities" read before the Society of Municipal Engineers of New York, made a distinction of this character, as indicated by the following quotation from that discussion:

"The great interstate public carriers are essential to the general development of the country as a whole, and make possible the assembling of the population in widely separated communities and the supply of those communities with all the necessities of life which surrounding territory produces and the comforts and conveniences which modern city development demands. This mutual dependence between a common carrier and its particular dependencies is also manifest in a somewhat different manner, but none the less important, between two cross country railroads which may be serving independent territories under non-competitive conditions, but which are truly competitive as to coast to coast traffic. The one railroad may pass through more non-productive country than the other, the one may be capable of a greater carrying capacity per ton-mile by reason of superior grades and alignment than the other, and the one may have superior coast

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terminals and may have cost much more to construct than the other.

"The cost of these two lines of road is one thing, but the value, the earning value of each, is quite another thing. One railroad may have adopted a policy of building quite as much for the future as for the present and may have a much greater investment mile for mile than its competitor which may have been built more with reference to present requirements with the expectation of improving structures, alignment and grade as its traffic increases. The one road may be said to have been overbuilt in the language of the paper under discussion, but this fact should not in any way militate against present value. In a rate case values of the carriers, which are competitive as to through traffic and non-competitive as to local traffic, should be so adjusted that each can earn a fair return though not necessarily at the same rate, and thereby be enabled to give proper service to the territory dependent upon each for business prosperity and general development."

To the statements just quoted the writer would add that, even in the case of competitive properties like railroads, there may be found less difference than is presumed to exist between cost, properly estimated, and value of the individual property, regardless of the wide variation of structural costs as between independent properties, because the earning capacity of the respective properties, being dependent on operating as well as structural conditions, may admit of such an adjustment of competitive and non-competitive rates that each property may earn a fair return on a value based largely or wholly on reproduction cost considered in its broad sense.

The cost of a public utility property, fairly determined by whatever method or methods, represents a state closely approximating the facts, and whenever an appraiser departs from his belief as to fact, on the presumption that cost is not value, and attempts to reason from cost to value, he enters the realm of speculation, to a greater or less extent, and will experience difficulty in finding a well-defended or defensible landing place. In fact, there will always be at least two conceptual values, one above, and the other below, the reproduction cost, to confuse in reaching a definite result through such a line of reasoning.

Although the question of determining value possesses business aspects and requires consideration from the business man's point of view, still the determination of value for purchase and sale is separate and distinct from that of value for rate-making. In other words, commercial value, the business man's value, is not necessarily earning value, and *vice versa*. For instance, in attempting to reason from cost to value, it may be argued that reproduction new, under present-day conditions, should take into consideration the experience gained in the construction of the existing property, and, reasoning from this general proposition, occasion can readily be found, as in valuing a water-works, to substitute an equivalent single main for two or three

existing mains, a new water supply from the same or a different source at a less cost than the existing water supply, new and modern pumping engines for old and non-duplicatable machines, and so on down the line for each item of the property, finally to arrive at a value of a substitute plant capable of rendering service as efficient as the existing plant at a cost considerably below that of reproducing an identical plant. Would such reasoning be just? Mr. Kiersted.

Take a further example of a town which, through loss of certain business resources, may have become to a degree depopulated, or that of an owner of a public utility property who, in anticipating future requirements, may have built in advance of present-day requirements. The investment in each case is a fact; but the situation that a town may have become to a degree depopulated, cannot properly be advanced as an argument for depreciating the original investment, judiciously made, any more than it can be advanced as a proper reason for an increase of rates. It is clear, however, that the rate of return on the investment must be less than it was before the town became partly depopulated. Suppose the author's views with respect to reasoning from cost to value obtained in this case, and the original investment was depreciated 50% because of the depopulated condition of the town, and a 7% return was allowed on the depreciated value, and that thereafter the business of the town revived sufficiently to restore the original population conditions without any addition having been made to the property, and in the meantime the net earnings having doubled—would a public service commission, on the one hand, be justified in cutting the rates on petition of the rate-payer, or, on the other hand, on petition of the owner, could the commission find satisfactory reason for removing the excessive depreciation formerly applied and thereby make good the actual investment, in order that the owner might receive a fair and adequate return thereon? The case of the owner of an overbuilt property is a parallel one.

It looks to the writer as though the author, in introducing the "cost to value" line of reasoning, is confusing the process of rate-making with that of valuation—two independent processes, each complete in itself, in natural sequence.

The fact is that a public service commission, having received or made complete estimates from several viewpoints of the cost of producing an existing property, and having compared the same with the accounts of original cost to the fullest extent possible, and finally having reached a conclusion with due allowance for interest, the cost of financing and development costs, has accomplished all that can be expected to be accomplished in arriving at a fair earning value of the property. The commission then proceeds, by an entirely independent process, to determine a schedule of rates which will yield

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an income duly providing for operating expense, repairs, general maintenance, replacement fund, operating contingencies, and a return to the owner, in such an amount and at such a rate as, under the particular circumstances, will be just and equitable to both rate-payer and owner. In this process of rate-making the commission, not being confined to some specific rate of return, can provide for such reward as good management and good engineering may be entitled to, and can make such discrimination between the rate of return on overbuilt property and underbuilt property about to be improved, as may be fair and equitable, and thus avoid unnecessary and phantom distinctions between cost and value. With these considerations in mind, it can scarcely be conceived necessary or advisable to exclude all thought of value during the detailed process of valuation of a public utility.

Mr. Burns.

CLINTON S. BURNS,* M. AM. SOC. C. E. (by letter).—The writer is impressed with the broad-gauge treatment that the author has given to this subject, and wishes to add his endorsement of the fundamental principles therein presented. He wishes not only to add his endorsement, but to add emphasis to one point at least touched on by the author regarding the relationship between cost and value. An endless amount of confusion seems to be caused by many writers failing to grasp the truth that cost is not value, and indeed may have no relation whatever to value. This fact may perhaps be aptly illustrated by an incident from the experience of the writer in acquiring possession of a supposedly valuable puppy, 3 months old. The former owner priced the pup at $25, and convinced the writer that this was the value of the animal, whereupon ensued a transfer of cash and ownership of the pup. On taking possession of the beast, his first official act was to pick up and devour a $20 bill, the cost of the pup automatically nearly doubling without any visible increase in value. In fact, there was at that particular moment much doubt in the mind of the writer whether he really possessed any value whatever, but, as the months passed, the pup developed into a respectable dog, at a cost about normal for this class of development expense, and has since taken prizes more than sufficient to pay his development expenses, including the original purchase price and the initial $20 feast. His value at the present time is considered to be in excess of, and entirely independent of, his past cost.

Public utilities are very similar in many ways to this dog. Their development expense may begin with a startling expense account, entirely unlooked for, after which they perhaps settle down to a normal development, and steadily appreciate in value. Then, when the time comes for an appraisal of the utility, the owner is confronted with the possibility of having its value judged by an appraiser who

* Kansas City, Mo.

may hold the view that past cost represents value, a theory that would clearly disqualify its advocate from holding a position of judge at a dog show. Mr. Burns.

Passing now to the "conceptual process" outlined by the author: the writer is impressed with the statement that much of the cost of reproduction will depend on the time assumed for the practical operations. This is particularly true of many of the owners' expenses for organization, administration, interest during construction, and similar items, as it is the writer's experience that the time usually assumed for construction work is much too short, due to the time that it has actually required in much of the larger work with which the writer has been identified. This is especially true as to the time required for preliminary investigatons and financing of public enterprises.

Another point occurs to the writer as worthy of special emphasis is suggested by the author's "summary of reproduction". He states:

"Ninth.—If a plant is carefully and honestly built, if it serves the need in the community in the best possible manner, and if there is no better or less expensive service, then the cost of reproduction must of necessity be the least possible value that can be put on the utility, but the real value may often be in that case more than the reproduction cost."

This statement offers an opportunity for a serious consideration of a far-reaching point, affecting not only the reproduction cost of a public utility, but in many cases its past cost as well. It is sometimes argued by members of the legal fraternity that certain portions of the property of an over-built plant are not properly included in a valuation, on the theory that the law limits them to a consideration of property used and useful. Hence, they argue, some of the real estate perhaps held in anticipation of future requirements, or an extra liberal provision in plant capacity in a rapidly growing community, not being used or necessary in the plant at the present time, is not properly included in the valuation. Now, if it be granted that such an argument is sound and in accordance with the proper interpretation of the law, it occurs to the writer that it must follow that the allowance for interest during construction must be correspondingly increased, as shown by the following example illustrating the case:

Assume that a property is found to be 25% over-built, and wisely so, in anticipation of future requirements. For example, let it be assumed that a property is found to have a reproduction cost of $1 250 000, of which amount $1 000 000 is all that is required for present use, but the additional $250 000 will be called into use and required to serve the public, say, 5 years hence. In nearly all communities where such a condition exists, it is a fact that the public utility property has passed through a similar condition throughout its entire history; therefore, when the whole property comes into use 5 years

Mr. Burns. hence, its reproduction value should include an extra allowance of 5 years' interest during construction on this $250 000 investment, because, from the very nature of the investment in the growing community, this cost had to be incurred 5 years sooner that it could be put into use. This $250 000 investment, when finally transferred to the used and useful class, therefore, should include 25 or 30% additional for interest during construction. Therefore, in estimating the cost of reproduction of such a property, where the conditions of growth are such as to warrant the belief that the property should keep 25% in advance of the immediate demands of the public, then this extra allowance for interest during construction must be included in any estimate of the reproduction cost. Similarly, this being true of the $250 000 investment now being questioned by the aforesaid attorneys, it is likewise true of the whole $1 000 000 property now admittedly used and useful, this entire $1 000 000 property having been similarly built piecemeal, and maintained 5 years ahead of time, all in the interest of the public welfare. This is a point which, as far as the writer is aware, has never been presented in any of the literature on appraisal and valuation work that has come to his attention. However, it occurs to him that, in a situation where the requirements of the public seem to demand that the public utility property be kept pretty well in advance of the times, this question of interest during construction for a period of time equal to the average over-built condition of the plant should be allowed in all estimates of cost of reproduction, and is, therefore, an element of value to that extent.

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THE VALUATION OF PUBLIC UTILITY PROPERTY.

Discussion.*

By Joseph Mayer, M. Am. Soc. C. E.

Mr. Mayer.

Joseph Mayer,† M. Am. Soc. C. E. (by letter).—Mr. Gandolfo's paper represents a widespread state of feeling toward public utilities, which influences the action of many voters and public men. The idea that compensation should be in proportion to the effort made to secure it is the leading motive of his paper, and of the bulk of the socialistic working classes, as well as of many of the most unselfish philanthropists.

In the competitive system of the production of wealth, actual profits of different industrial enterprises are hardly ever average profits, and, instead, vary widely from losses of part or nearly all of the invested capital to very large annual returns, even exceeding the investment.

These differences sometimes result from unregulated natural monopolies (and have then often little or no connection with superior industrial skill), but more often from differences in the industrial talents of the leaders of the various enterprises. These differences are purely mental qualities, which cannot be measured directly, but can only be judged from the revenue which they produce.

The promoters, managers, and owners of such enterprises consider them as tools for obtaining a revenue, and they endeavor to create these tools at the smallest possible cost. The value of the tools depends on the revenue they secure. Though their cost of production, including average profits, has some influence on their value, the most cursory observation of the actual returns secured on the invested capital of different competitive enterprises shows that there is no definite

* Continued from January, 1915, *Proceedings*.
† Montreal, Que., Canada.

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or close relation between the cost of production of industrial enterprises and their revenue and value. Therefore, those who attempt valuations of complicated enterprises by ascertaining their cost of production meet with various unsolvable problems, especially when they try to ascertain the proper compensation for the mental work of their leaders. They often complain of the excessive compensation of the promoters and financiers who are the middle men between the investing public and those who first suggest or conceive the enterprises.

One of the most difficult and important tasks in the creation of an enterprise is to ascertain whether it should be created, or whether its value when finished will equal or exceed its cost of production. It is evidently desirable to use the available resources for the production of the most useful enterprises. The main function of the middle men between those who first suggest an enterprise and the investing public is to suppress the unprofitable and advance the profitable enterprises. Bankers who offer to the public such securities as prove profitable investments gradually acquire customers who will buy the securities they offer, guided mainly by reliance on the bankers' judgment and honesty. If the bankers possess the ability to judge correctly the value of the securities, or the income which they will bring, and the current relation between revenue and market value, and if they issue them at a profit and below their market value, they perform efficiently the most important and difficult function in the creation of industrial enterprises. These bankers either buy these enterprises from the promoters and then sell them to the public, or they sell their services to the promoters for a commission. These services essentially consist in investigating the enterprises offered, in thereby forming an approximately correct idea of their cost and future net income, in ascertaining the commercial value of the securities issued, and in offering them to the public with their approval. The promoters take up ideas suggested by inventors of such enterprises, assist in the creation of detailed plans and estimates of cost and revenue, and in obtaining the favorable opinion of trusted and well-known men of good judgment on such matters. They then present to the bankers the facts and arguments which assist the latter in forming an approximately correct opinion of the cost and probable revenue.

The attempt to fix a proper compensation for either the promoters or bankers by prescribing a percentage on the cost, or a salary which should not be exceeded, must inevitably fail in properly adjusting it to the value of the services rendered. This value depends mainly on the intelligence, experience, honesty, and reputation of the workers, and only to a slight extent on the work spent on the task in hand. Any meddling with the compensation secured under free competition, beyond insisting on complete publicity and the honesty of all the

accounts, must inevitably be rather a hindrance than an aid to the attainment of justice. Practically the same applies to attempts to prescribe the salaries of the leaders of industrial enterprises. The supreme aim governing all the activities of the creation and management of a competitive industrial enterprise is to secure the largest revenue at the least cost; and of thereby making the difference between the value and the cost of an enterprise as large as possible. As long as there is free competition for the purchase of materials and the services of men, and for the sale of products, the compensation of the owners and the leaders is kept within just limits, and the widely different profits of the enterprises reward those bond and stock buyers who used their resources in a judicious manner for the welfare of the community, either by forming a correct opinion of the reliability of the judgment of others or by following their own good judgment; and punish those who wasted their resources on enterprises which would have been better prevented. If the profits were the same for all investors no adequate effort would be made to choose profitable and avoid unprofitable enterprises. The public would then have to pay average profits on the investments in injudicious enterprises, and would get the excess over average profit of the extremely small number of judicious ones which, under such conditions, would have any chance to succeed. The present liberty of investment would become utterly impracticable, and a public authority would have to be created to select the enterprises to be undertaken. Exactly the same holds true for every important step in the management of an enterprise in operation. Public management would be inevitable. Mr. Mayer.

The competitive system without efficient public control has produced in the course of time values which have, especially for old enterprises, no definite relation whatever to their costs of production to date. To value enterprises which have grown up with little or no public control on the basis of their cost of production to date would cause such a wholesale confiscation of the real value, dependent on revenue, of some, and such an enormous increase in the value of others, that the resulting storm of protest would sweep any commission attempting it into disgraceful oblivion. To imagine that any Court would sustain such a valuation exceeds the capacity of any one with the least knowledge of the nature of competitive business. The author thinks that an arbitrarily chosen or guessed at going value should be added to the physical value in order to obtain the total value of an enterprise. Why not guess the total value without going to the trouble of ascertaining first the physical value? Equally reliable results could be obtained thereby.

It is evident to all competent observers that the necessity of public control of prices arises from the disappearance of competition. Whether the monopolists are private persons, partnerships, or stock

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companies with few or many stockholders, is entirely immaterial. The author neglected to give any definite advice in regard to the control of issues of securities. He evidently intends to make the securities cover the actual costs incurred, but his description of how to do this is partly indefinite and partly unreasonable. He claims to show that cost to date should govern rates, provided there has been no fraud, gross mismanagement, or extravagance.

This cost, however, is to be only the basic figure and, as he asserts later, a guessed at going value should be added. The writer is of the opinion that Mr. Gandolfo has not proved his case.

He also claims to show that the same principles should be applied to the valuation of the property of a private individual. What sane buyer or seller of any business will be governed by its ancient history, except in so far as it throws any light on its future earnings? All sane transactions of sale between competent judges of values, able to follow these judgments, which alone throw a light on the real values, are governed by the benefits the owner can derive from the property after buying it. The author, however, is obsessed by the theory that the intelligence used in the creation and management of an enterprise deserves no consideration whatever as a cause of value. He also neglects to describe clearly the method by which cost to date, which is an important fact in ascertaining the rate of profit on the investment obtained by an enterprise, can be ascertained correctly.

Although the writer believes that the paper, as a logical product, is without value, it expresses an influential state of public feeling which demands a reduction of the prices of monopolized products and objects to the persistence of a condition which gives to the investors in monopolies larger profits than are obtainable in competitive enterprises; any argument or utterance which promises some way out of the present condition is welcomed by a large class of readers, because it agrees with their feelings. A way out must be found to obtain justice and thereby to satisfy those who keenly feel that the present condition is unjust. The past franchises are mostly indefinite and often unduly favorable to the owners of monopolies, but they often contain a clause prescribing that the rates must be reasonable. Reasonable rates are evidently such as will give to the owners of monopolies the same profits as are obtained in competitive enterprises, and a regulation of the prices which secures this result is just, where such a clause exists and where it has not become obsolete by neglect to enforce it. The terms of the franchise, therefore, must be consulted, and if they are not obsolete, what has been promised governs the regulation which fixes the rates and the consequent value of the monopoly. The public, however, has the right to expropriate from the owners by paying them the value of the monopoly resulting from this regulation.

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If another regulation than that originally intended has been legally introduced and maintained, then the value of the monopoly, with this other regulation legally in force, must be paid to the owners. What kind of regulation is legal at present is a question for judges and legislatures to decide, and is a necessary preliminary to any valuation of natural monopolies. After this legal regulation is known, and has been in force for some time, the probable future net earnings with it can be judged from the recent ones and the causes of change.

The owner of the monopoly owns the future net earnings, with the present legal regulation. These future net earnings have a definite present value which can be ascertained as soon as they and the proper interest rate are known. The proper interest rate is that rate which makes the present value of the future earnings of similar enterprises equal to their market value. There are some enterprises which have a fair market value and approximately known future earnings; the proper interest rate, therefore, can be calculated. The present fair value of monopolies, therefore, can be closely estimated without any circular argument, and without the use of any arbitrary guessed at going value.

After the present fair value of the monopolized enterprises has been thus ascertained from the estimated future revenue, with the present legal regulation, it is possible to establish such a regulation as will give to the value thus ascertained, and to all future investments, merely competitive profits, by a method previously fully described by the writer.

The author's notion as to how profits and values are ascertained in a competitive business appears preposterous to any one familiar with competitive enterprises. The products of such enterprises are sold at what they will bring in the market. The difference between the price obtained and the cost of production is the net revenue secured. The manufacturer endeavors to obtain with his plant the largest possible net revenue in excess of interest on borrowed money. He will extend or diminish the scale of production of the different articles or services he furnishes so as to secure the largest possible net revenue. He may go out of business when he finds he can obtain better total compensation for his time and property by selling his business and doing something else, but as long as he is in business he has no choice except to take what revenue he can get, which, especially if his business is old, is almost altogether independent of its cost to date. The value of his business is evidently the present value of the future net revenue which can be obtained from it. This will be estimated from the recent net revenue, less a proper salary for a managing owner, and the causes of future changes of net revenue, among which the cost to date, if the business is old, is one of the least important. This competitive method of ascertaining the prices

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of the products and the value of a business, which any one not quite ignorant of competitive enterprises will recognize as the actual one, is evidently radically different from that advocated by the author for public utility corporations.

When a nation, a state, or a city, engages in business, it is, like any other owner, interested in the profits or losses resulting therefrom. The cost to date must then be ascertained to find the rate of profit on the investment. This rate of profit secured during a short interval of time, however, cannot be ascertained accurately. The depreciation must be considered as a part of the operating expense which is subtracted from the gross earnings to obtain the net earnings. The actual depreciation of any part of an enterprise which occurs during its life, is its original cost less its salvage or scrap value when discarded. The distribution of this depreciation over its life is at best arbitrary. The depreciation during any one year, therefore, is unknown, even after the article has been discarded.

The depreciation of all the parts still in use is known still less accurately because their life is unknown. The remaining value of an old part without scrap value when discarded, which renders equally good service during its life, is often given as its probable remaining life, divided by the probable life of the part when new and multiplied by the original cost. This is not true, because it neglects interest, which is a very important factor, especially if the life is long. It is also never true that a part renders equally good service during all its life. Inadequacy and obsolescence enter as other unknown factors in depreciation. As the actual depreciation of the parts of a business enterprise during any one year cannot be ascertained with accuracy, the same is true of its net earnings. When, in ascertaining the investment, depreciation is only subtracted when a part is discarded, the investment is over-estimated by the amount of depreciation of all the parts still in use. Although, theoretically, depreciation should be charged when it occurs, practically, whenever there are no expensive parts, it is often best to charge it when the parts are discarded, as then the depreciation of the discarded parts is first accurately known. When comparing different enterprises, it is important to use in all the same method of allowing for depreciation. The author's remarks on how to ascertain the cost to date are rendered quite indefinite, as are all his reasonings, by lack of clear definitions of any of the terms he uses.

That the value of an industrial enterprise is not given by either the cost to date or the cost of reproduction less depreciation, is perceived by all close students of the problem, and becomes apparent whenever the attempt is made to define the terms clearly, and describe accurately either method, and to apply it to concrete cases.

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Franchise value and going value, are the expedients used to bring the proposed methods into agreement with the facts.

When a franchise gives the right to charge rates which give more than competitive profits, it has a value. This value is the present value of the future excess over competitive profits which the franchise allows. It must be estimated, therefore, by studying the terms of the franchise, ascertaining the consequent excess of future profits over competitive ones, and their present value. To make such an estimate possible, the franchise must define the regulation. If it can be shown that a franchise is against the public welfare, it may be invalidated by the Courts. The value of a franchise, therefore, is largely a legal question. It is not equal to the cost of obtaining it, as the author intimates.

Going value has been defined as the difference between the value of a business and its physical value. Some other method of finding the value of a business than the cost to date or the cost of reproduction less depreciation, therefore, must be found. This method, more or less hidden by complicated verbiage, is to capitalize the revenue.

The following equations summarize the method: If T is the total or market value, G the going value, and Ph the physical value, then

$$T = Ph + G \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where

$$G = T - Ph \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

From Equations (1) and (2),

$$T = Ph + T - Ph \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

or,

$$T = T \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The argument is: To find the total or market value, add the going value to the physical value, Equation (1); to find the going value, subtract the physical value from the total value, Equation (2); and to find the total value, add and then subtract the physical value from the total value. This is the method followed in substance, with much intricate phraseology, in a recent paper in a prominent economic journal. Mr. Gandolfo rightly prefers to guess at the going value. By this process, it is always possible to obtain any desired result, and, therefore, it is a very convenient method.

The problem of the valuation of natural monopolies and the regulation of the prices of their products is of the utmost importance at the present time, and the engineering societies can render a great public service and raise themselves in the public estimation by discussing it intelligently and thereby contributing to its solution. Outspoken criticism of popular errors is a

Mr. Mayer.

necessary part of such discussion, and, the writer believes, should be without reserve when one succeeds in getting a hearing before a scientific society.

The writer is of the opinion that this paper does not show the essential realities of the problem, that its definitions are not clear, and that it does not give an accurate description of any method of valuation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

INVESTIGATION OF THE PERFORMANCE OF A REACTION TURBINE.

Discussion.*

By Daniel W. Mead, M. Am. Soc. C. E.

Mr. Mead.

Daniel W. Mead,† M. Am. Soc. C. E. (by letter).—The author has introduced a new method, new, at least, to the writer, for an approximate analysis of turbine characteristics from a very limited number of tests, which is, the writer believes, of considerable value. It must be recognized, of course, that this method is approximate, for the three experimentally determined points through which the author has drawn the torque-speed curves (Fig. 10), could lie equally well on several other curves which could be drawn through these points. The accuracy of the results will depend largely on the experience of the investigator. The extended analysis from the limited data of these tests well illustrates the greater value which could be secured at the Holyoke Testing Laboratory, if that Laboratory would uniformly add to its regular tests, the determination for each gate-opening tested: first, of the torque at that gate-opening with the wheel at rest; and, second, the runaway speed at such gate-opening with the brake removed.

Many of the Holyoke tests are confined to a few experiments near the speed of maximum efficiency, and these experiments are frequently so limited that no deductions can be safely drawn as to what may be expected of the turbine tested if it should be operated at a speed somewhat different from the most efficient one. The value of the tests, both to the manufacturer and to the engineer, would be greatly enhanced, in the writer's opinion, if tests for torque at no speed and for runaway speed were added and, if necessary, fewer tests were made

* Continued from January, 1915, *Proceedings*.
† Madison, Wis.

Mr. Mead. under the conditions of speed approximating those of best efficiency. It is obvious, of course, that the more test points actually determined, the better; but, if any are to be omitted, they should not be in general those at the extreme conditions of speed, for such tests are the limits which largely define the characteristics of the turbine.

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PAPERS AND DISCUSSIONS

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PENSTOCK AND SURGE-TANK PROBLEMS.
Discussion.*

By J. T. Noble Anderson, M. Am. Soc. C. E.

Mr. Anderson.

J. T. Noble Anderson,† M. Am. Soc. C. E. (by letter).—That it may be known what degree of dependence can be assigned to the formulas, and the empiric figures and factors therein, which are given in this paper, the writer would like additional information on the following points:

First.—What is the authority for giving the modulus of elasticity of water as 42 400 000 lb. per sq. ft. (this being practically the same as that determined by Colladon and Sturm)?

Undoubtedly this modulus varies greatly with the density (purity) of the water and with the temperature (Grassi). Of course, if the author can show that this value is a minimum one, as seems likely, that will suffice, because obviously the minimum elasticity of the water means the greatest effect of the water-hammer or surge.

Second.—What is the experiment or series of experiments relied on in fixing 4 000 000 000 lb. per sq. ft. as the modulus of elasticity for steel plate when worked up into a conduit, which must be to some extent discontinuous, both on account of the necessary expansion joints and the imperfections of riveting or other manner of jointing? Then, even assuming that it is sufficiently established by experiment that these apparently disturbing elements make no appreciable difference (as has been claimed repeatedly), is not this modulus rather too low for modern steel plates. Obviously, for the same reason that it is safe to use the lowest modulus for the water, it is only safe to use the highest possible modulus for the steel.

* Continued from January, 1915, *Proceedings*.
† Narbethong, Victoria, Australia.

Mr. Anderson.

In the case of wood, it would seem likely that a double formula is required, because the thickness of the walls of the conduit—greater than a certain critical thickness—would entirely damp out the elasticity of the steel bands as a determining factor.

One other question, based on the writer's experience in pumping through long conduits: Does Fig. 2 represent, at all accurately, what actually happens? Will not the regularity of the recurrence of pulsations be disturbed by numbers of minor harmonic waves?

These minor harmonic waves are of more than academic interest, as the smallest appreciable disturbance in the load may often cause financial loss.

In conclusion, the writer notes that most of the symbols used by the author are in accord with those of well-known hydraulic writers, but one or two are disturbingly unfamiliar, namely, a = velocity of vibration along pipe, in feet per second; and e (instead of Greek ϵ) = 2.718, base of Napierian logarithms; and he would like to emphasize the desirability of adopting a universal nomenclature and set of symbols for the whole hydraulic engineering world.

In 1908 the writer urged the introduction in Australia of the name "cusec", which had long been used in Anglo-India for the now universal unit of flowing water, the cubic foot per second; and the acre-foot, then used in America, as the most convenient unit for large irrigation storage reservoirs. It is gratifying to know that since that time these terms have been generally adopted in the irrigation parts of Australia.

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PAPERS AND DISCUSSIONS

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SUBMERGED PIPE WORK AT PORTLAND, OREGON.

Discussion.*

By Messrs. Thomas H. Wiggin, W. E. Spear, John P. Hogan, and W. W. Brush.

Mr. Wiggin.

Thomas H. Wiggin,† M. Am. Soc. C. E.—The author refers to the scarcity of information about submerged pipe lines of the larger sizes. In connection with a 36-in. line which the New York City Board of Water Supply is laying across The Narrows, the speaker had occasion to look up articles on submerged pipe lines, and can vouch for the author's statement, particularly with reference to the completeness of the records. Thanks are due to the author, and to Messrs. F. and A. S. Riffle, who wrote the previous paper, for supplying such valuable information on the large Portland submerged pipes. The speaker would suggest, however, that Mr. Clarke make a few additions which will fill some of the wants felt by the speaker when looking for data on such pipe lines.

Prices, for example, are given sometimes in lump figures. A price per pound, or details permitting easy translation to price per pound, would be of much more general use. In connection with the dredging, the price given is so much per cubic yard of material. A question immediately arises as to how the material was measured. One of the main problems in specifying for The Narrows siphon was how wide the bottom of the trench should be, what the side slopes should be, and whether dredging should be paid for by place measurement or by scow measurement; in other words, whether a great deal of

* Continued from January, 1915, *Proceedings*.

† New York City.

Mr. Wiggin.

latitude should be allowed in the width of the trench, and the City be made to take the risk as to whether the quantities would over-run, or whether a particular width should be prescribed and merely theoretical quantities be paid for.

There is also the question of the uniformity with which the bottom can be dredged. For example, is it practicable to dredge the bottom within 6 in. of a given plane, as has actually been attempted, or must an allowance of several feet be made, so that the pipe line will go up and down, which, being a flexible line, it is able to do? Those details, as to the actual success in doing the work, would be helpful.

The exact method of making up and caulking the joints is of great importance. The joint used at Portland is what is commonly called a Fanning joint, from the originator of the type. It will be noticed in the illustrations that it is divided by a flange on the great circle. That enables it to be made up in two parts; in other words, the bottom of the joint can be poured and caulked, and then the outer part of the joint can be treated in a similar manner, thus producing two zones of compacting for the lead; that is, two rings of caulked lead. It is important to know how the work was actually done.

The extent of under-water caulking is also a very important matter in a long line of this sort; that is, was it generally made permanently tight at the surface and then sunk to its position in the trench, or was it caulked again after it was submerged. This touches on an important point, namely, how well a flexible joint will stand being deflected, and how tight it will remain after it has been deflected. In some of the experiments in connection with the development of a design for The Narrows siphon, caulked joints were tested before deflection, and then deflected and tested again, and it was found that the movement of the joint caused it to be about as leaky as it was before it was caulked originally.

The particular type of joint used at Portland may be more successful in that respect, if it was caulked in two parts; for example, if the center part of the joints—that part on the great circle—is actually caulked. The joint tried on The Narrows siphon experiments was not caulked on the great circle, but at the outer point of the joint. It was the so-called Duane or modified Ward joint.

The depth to which caulking extends is surprisingly small. In some experiments by the Board of Water Supply, portions of a joint like the Duane were made up with pieces bolted together, so that after caulking they could be unbolted and taken apart; it was found that, in a joint about 6 in. deep, the only part that was affected by very vigorous caulking with pneumatic hammers was about $\frac{3}{4}$ in. at the face; the remainder of the lead was so far from being in contact that a small wire could be run between the lead and the iron. It is evident, therefore, that the method of caulking and the location

of the zone on which caulking is done is very important, and particularly the question as to how much caulking will be required under water to produce a finally tight line. Mr. Wiggin.

Reference has been made to the effect of pull on joints, and it will be perfectly evident, the speaker thinks, that a pulled joint is bound to be tight while it is being pulled; the condition is that of a sphere being pulled out of an enveloping sphere which is too small at the opening to permit the escape; and the escape is being prevented by a band or zone of lead which is squeezed between the inner and outer spheres. The harder the squeeze the more water-tight the joint will be. The question is, will it stay tight after the pulling and resulting squeezing have stopped?

The ideal line might be one that is laid on such a foundation, or say, such lack of foundation, that it will at all times be subject to a catenary pull by virtue of a slight tendency to settle. Such a line would remain tight. The practical difficulty of obtaining such conditions is obvious, though it is probable they exist in some short lines; but if one imagines a line laid on a level bottom, and made tight by pulling, and then supposes that the temperature of the water surrounding that pipe, and in the pipe, as might be the case, rises 30 or 40°, and each length of pipe lengthens, then the contact would be relieved. The same condition would result from settlements, even slight, at accidental local summits in the line or at changes of grade where the line was convex upward. The only thing that would keep the joints in contact would be the elasticity, not in the lead (which has practically none), but in the iron inside and outside of the joint; that is, in the bell and spigot. If these are stressed sufficiently during the pulling of the joint, and if it is supposed (which in the light of experiments by the speaker is improbable) that the lead will not yield gradually and relieve that stretching and compression in the iron, then the change in length of the pipe due to temperature changes might be compensated for by a contraction of the bell, thus keeping tight the joint which had been made tight by the stretching process.

None of these questions, however, has been answered from existing literature, because the behavior of submerged lines after having been placed in service is not known; in other words, there are no meters of adequate kind—in fact, generally speaking, there are no meters of any sort—to tell whether the lines remain tight. Only the grosser leaks can be discovered, such, for example (like that described by the author), as would lower a reservoir.

In the pipe which is being laid in The Narrows, the attempt has been made to produce initial tightness, and to overcome the difficulty due to changes in length of pipe with temperature, by substituting for caulking the filling, by injecting lead with screws, of all shrinkage

Mr. Wiggin.

space in the joint so far as practicable, and especially on the great circle; then, no matter if the spigot is moved slightly in and out, the joint will still be tight on the great circle, and act as a sort of expansion joint, rather than be tight at the front only and afterward become loose by temperature movements or movements due to settlements in the line.

In connection with the break in the 28-in. pipe and the uncertainty as to the location of the break, the thought occurs that perhaps it would pay to put meters at each end of such a line, in order to discover when it starts to leak more than a slight percentage. A check-valve between the crossing and reservoir would have prevented the emptying of the reservoir. Of course, it may not always pay to make such provisions. On the line across The Narrows, meters and check-valve both have been provided, because it is very important that it should be known if the line starts to leak, and also that the water stored in the reservoir should be conserved for use while the line is being repaired.

The break in the Portland 28-in. pipe took place by fracture across one of the pipes, which are nearly 16 ft. long. The paper gives no diagram, but, from the description, the break took a long slant, such as would occur in a cross-grained stick of wood, the fracture being apparently about 12 ft. long. This would indicate that the pipe was perhaps resting on a hard spot in the middle and acted as a beam. Some such explanation is evidently in the mind of the author. It raises the question as to whether 16 ft. is not too long for a cast-iron pipe, particularly on a hard bottom. In the case of The Narrows siphon, it was decided, after considering the Portland 28-in. pipe, which is very well described in the paper by Messrs. F. and A. S. Riffle—a paper which furnished a great deal of useful information in connection with studies for The Narrows pipe—to adhere to the shorter length of 12 ft., partly because foundries are rigged for 12-ft. lengths, but mostly for greater safety in connection with foundations and flexibility.

The new Portland steel lines with cast-iron flexible joints have considerable stretches between flexible joints, apparently more than 35 ft., and the question suggests itself whether the pipe will continue everywhere to span successfully between flexible joints, particularly if the bottom is hard, and in view of the fact that the back-fill under a pipe is never perfect.

Experience with The Narrows pipe proves that it is very difficult to get a level bottom, though the method of dragging the trench bottom is now producing good results. At best, however, there will be occasional sharp changes in grade, due to the working of the pipe-laying cradle in rough weather. Back-filling at such places is likely to cause undue strains in joints and in pipes. Further details of the

depth and method of placing back-fill at Portland would be interesting. A depth of from 3 to 4 ft. is mentioned. A photograph in the Riffle paper shows a comparatively deep trench at the shore. Mr. Wiggin.

The details of the repair sleeve are very interesting; the speaker, however, would like to have a statement as to how the sleeve was filled. Apparently, it was grouted after being placed. The 8 by 8-in. angles and other parts of the longitudinal joint, if they were required to take the full hydrostatic pressure over the full diameter of sleeve, would apparently be very heavily strained. Was the grout assumed to be so impervious that the pressure did not reach the whole area of the sleeve?

Another point, on which information would be useful, is the accuracy and smoothness of the machine work. In the original studies for The Narrows pipe, much help was obtained from the records of the 28-in. Portland line in the matter of machining, and an extension of the record to include the new lines would be valuable. In the matter of smoothness, no accurate data were found, because it is difficult for civil engineers to describe machining. It is not impossible, however, to describe the lathe tool, speed, feed, velocity of revolution, etc.

The success of a flexible joint, to a large extent, depends on how smooth the turned parts can be made. In the work now being done for The Narrows line, not due to the specifications of the city, but to the contractor's energy in the matter and to his appreciation of its importance, the surface is actually being ground, making it equal to good cylinder work, and the results are apparently justifying the method, although, of course, as the line has been only partly laid, it is too soon to say what the final outcome will be.

In experimenting for The Narrows pipe, lead wool, with the different degrees of caulking, was tried. It was easy to get a tight joint, but the trials were not successful in obtaining at the same time the proper degree of flexibility. The experiments were not wholly exhaustive, as the method was much slower than desired and the method of injecting lead mentioned previously proved to be more desirable for flexible joints.

Non-shrinking alloys of lead were also tried, by a company which has all the metals and is expert in mixing them, but it was impossible to mix one that seemed to meet the requirements. This idea looks very promising, and the speaker is now sure that he understands why it was not successful.

W. E. Spear,* M. Am. Soc. C. E.—Mr. Wiggin has brought out many of the points which this paper suggests to those interested in submerged pipes. It would be interesting to know how accurately the trenches at Portland were dredged, and also the final grade and align- Mr. Spear.

* New York City.

Mr. Spear. ment of the pipes. As Mr. Wiggin states, the 36-in. pipe line under construction in New York Harbor was laid at first on a somewhat uneven bottom, but the vertical deflections of the pipes were not great. In addition, the line also deflected laterally at some points, due to the thrust of the pipes in sliding down the launching ways. Similar deflections in the Portland pipe would have exposed that portion of the lead which had been caulked at the face of the bell, so that unless the joints were poured and caulked in two operations, as Mr. Wiggin suggests, it is difficult to understand how the line could have been made tight without submarine caulking after the pipes were finally placed.

Some experiments, made in New York Harbor several years ago, on the depth of penetration of large anchors, indicate that the cover of 4 ft. placed over the Portland pipes would not be quite sufficient. A cover of 8 ft. is being placed over The Narrows siphon.

In harbor waters the life of a submerged pipe line is a matter of some concern, and it would be of great value to those in charge of such works to know the condition of the 28-in. cast-iron pipe at Portland when it was taken out after 20 years of service, especially the condition of the finished surfaces of the bell and spigot, and how much work would have to be done on them to make them sufficiently smooth to place again in service. Apparently, they were good enough to permit them to be used again, because they are being stored for that purpose.

The work at Portland was evidently done at a very reasonable cost. The pipe line in New York Harbor is of greater size and is laid at a much greater depth, in a swiftly moving tideway, crowded at times with traffic. The cost of this pipe, on the basis of the quantities originally estimated for several items, would be about $95 per ft., but some economies will be effected in dredging and back-filling that will probably reduce the cost to about $85 per ft.

Mr. Hogan. J. P. Hogan,* Assoc. M. Am. Soc. C. E.—It is interesting to note that the pipes at Portland had to be taken up and relaid on account of the necessity of deepening the channel. In The Narrows siphon, which is now being laid by the Board of Water Supply of New York City, the Government requires the City to lay pipes so deep that they will never have to be taken up. The line is about 10 000 ft. long; the minimum depth will be 60 ft., and the maximum, 74 ft.

Mr. Herschel suggests that it might be possible to make up a line with riveted or screw joints and drag it across, but there would be some difficulty in doing that in this case, so that, even if a screw joint or riveted pipe is useful for short distances, a flexible pipe line has certainly a field of its own.

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* Tompkinsville, N. Y.

Mr. Hogan.

It would be of interest to know how long it takes to make up the Portland joint. In comparison with the Ward joint, which is being used in The Narrows, it seems to be rather clumsy, and might take a much longer time to make it up. In New York, the pipes have been made up and sunk in 1¼ hours per length, for the entire operation, and it does not seem possible to make up the Portland joint, which requires the bolting of a flange, in nearly as short a time, particularly if it is one with the double caulking. In any case it would be valuable to have every detail in regard to the exact method of making up this joint and the time required to do it.

It may be of interest to know that the joint used on the Board of Water Supply work is substantially as Mr. Wiggin has described it; the shrinkage of 10%, which occurs when the lead cools, is made up by forcing in cold lead through 32 holes by jib-screws. The joint takes about 300 lb. of lead at a pouring, and about 22 lb. are forced in afterward. With mechanical appliances, the operation of forcing in the cold lead takes about 30 min.

On the Portland joint the stop for holding the lead is in the bell, which would indicate that the lead remains fast to the bell and rotates on the spigot, whereas, in the New York joint, the lead remains fast to the spigot where it shrinks, and the bell rotates around it.

On page 2659* it is stated that "the final test, under a pressure of from 170 to 180 lb., showed a leakage of approximately 7 gal. per min., or 10 080 gal. per day, or about 8 gal. per lin. ft. of joint, per day, under the pressures noted, which are approximately 50% in excess of the normal working pressure." In New York, the experience has been that the higher the pressure the tighter the line, so that it would seem that, if the pressure was 50% in excess of the normal working pressure, the results shown are more favorable than would be found under the actual working pressure. In tests of a certain part of The Narrows line at 80, 90, and 100 lb., the leaks were greater at 80 than at 90 lb., and greater at 90 than at 100 lb. That may be due partly to the tightening of the joints at the ends of the line because of the longitudinal pull. With the New York joint, it is due, in a great measure, to greater resistance to pressure of the bell than of the spigot.

Both Mr. Spear and Mr. Wiggin have mentioned the up-and-down character of the New York line. That was a feature at the beginning of the work, because in dredging at a depth of 60 ft., it was difficult to do accurate work on account of the heavy tidal current in The Narrows, which is about 3½ miles per hour. It is also extremely difficult to ascertain the result of the dredging, because it is very troublesome to make soundings in heavy currents at a depth of 60 ft., and

* *Proceedings*, Am. Soc. C. E., for November, 1914.

Mr. Hogan. almost impossible to make area soundings. Fairly accurate profiles may be made, but it is very difficult to ascertain whether or not the bottom is even. In the first part of the line, especially in attempting to dredge down grade, the bottom was uneven, but that defect was obviated later by the use of a V-shaped drag consisting of two heavy steel members, about 25 ft. long, with a spread of about 20 ft. at the base, across which there is another heavy member with a plate inclined forward to form a cutting-edge. The weight of the drag is between 8 and 10 tons, and, by hauling it up and down the trench four or five times, it has been possible to get a very level bottom, so that at present no trouble is experienced from up-and-down variations of the pipe, except in storms. It is impossible to lay pipe during storms in such an exposed position. It is not safe to move ahead, and, at that time, the skidway or cradle in which the pipe is supported tends to work into the bottom, sometimes as much as 2 or 3 ft. That leaves a hollow or depression which sometimes causes a maximum deflection in one or more joints, but such variations in grade have been straightened out successfully. However, in laying at this depth and under these conditions, it is a question whether a satisfactory result could be obtained with any pipes but those with flexible joints, as one cannot expect to have the bottom of the trench as even as if it were dug on land or in shallow water.

In reference to Mr. Kellogg's remarks, it may be stated that the Ward joint, as used by the Board of Water Supply, has a bell which is large enough to receive the spigot, and has no flanges. Also, that no work is done under water. The joint is made up and made tight on top, and then lowered into place and never touched after it is laid.

Mr. Wiggin has stated that joints caulked with lead wool were tried. Undoubtedly, a lead wool joint can be made up successfully under water, but the effort was to get a joint which could be made up on top, and would be flexible enough afterward to be lowered into place and require no attention after it was laid; it was found impossible to do that with lead wool.

Mr. Brush. W. W. Brush,* M. Am. Soc. C. E.—Messrs. Wiggin and Spear have brought out the more interesting points in connection with the submerged pipe lines at Portland, Ore. An interesting question that has not been raised is how much deviation in line and grade is permitted in submerged pipe line construction? A study of the specifications does not give a satisfactory answer to this question, as they may or may not state definite limitations. Even when definite limitations are given, investigation has shown that the work, as carried out, did not come within such limitations. If Mr. Clarke could give this

* New York City.

information for these submerged pipe lines, it would be useful in considering specifications for similar work. Mr. Brush.

The experience of New York City in repairing submerged pipe lines is similar to that of Portland in regard to the time required. The officials responsible for appropriating funds do not seem to realize the necessity of a secondary feeder for safe-guarding a supply which is dependent on submerged mains. When the engineer states that the repairs to a submerged line may require weeks or months, rather than days, the layman frequently considers this statement to be biased. As a result, proposed pipe lines for secondary feeders are not put in, due to lack of funds.

Within the past year, New York City has had such a case, where one of two lines has been out of service for approximately 6 months, and will be for several months to come, and yet, less than 2 years ago, the authorities refused to authorize a contract for laying a secondary feeder main recommended by the engineers of the Water Department, although one line was insufficient for fire protection.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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HISTORY OF LITTLE ROCK JUNCTION RAILWAY BRIDGE, OF THE ST. LOUIS, IRON MOUNTAIN AND SOUTHERN RAILWAY COMPANY, OVER THE ARKANSAS RIVER AT LITTLE ROCK, ARKANSAS, 1883-1914.

Discussion.*

By Messrs. C. D. Purdon and F. G. Jonah.

Mr. Purdon.

C. D. Purdon,† M. Am. Soc. C. E. (by letter).—The caissons referred to in the paper were located correctly, and the masonry fitted as well as it generally does, it being hardly practicable to sink a caisson in exact location.

When the rock was cut level, as soon as a part of the shoe of the caisson reached it, all the rest was cleaned out and blocked up with timber before placing the concrete.

The cribs were filled with rock to the top before any sand was pumped in, for the purpose of giving weight to the pier, and not with the intention of carrying the weight. It is possible that in time, with the shock of drift striking the piers, etc., the rock may have worked under the cross-timbers and caused a settlement.

The only trouble with the piers was that they were designed for spans 14 ft. wide, in the clear, and the masonry was well advanced before the superstructure plans were received; these showed that the spans were to be 18 ft. wide, in the clear, and it was necessary to widen and lengthen the tops of the piers to get a bearing. Even then

* Continued from January, 1915, *Proceedings*.

† St. Louis, Mo.

Mr. Purdon. it was a tight fit, but the spans fitted correctly on the piers. All the piers were practically complete before the steel for the superstructure was received.

The piers were all built in accordance with the plans and specifications, and the batter was the same on all, except that the pier at the north end of the draw-span was built with its south face vertical, to shorten the draw-span.

Any difference that may have been observed in the batter later is readily accounted for by the crushing of the timber in the cribs; because of the narrow base of the pier, a small settlement to one side would be largely multiplied at the top.

Mr. Jonah. F. G. Jonah,* M. Am. Soc. C. E. (by letter).—The writer has read Mr. Smith's valuable paper with interest, and from personal knowledge of the conditions believes that the work was handled with great skill and by the safest possible methods. The author was quite right in insisting on a plan which would obviate the use of falsework. To carry a span on falsework over the Arkansas River for any considerable length of time is a hazardous proposition. The records show that serious rises have occurred in this river during every month in the year, and nearly every railroad which spans it has had trouble in the construction or renewal of bridges.

One other plan might have been used in a case of this kind, and that would be to sink a cylinder at each end of the old pier, up and down stream, place a cross-girder between, and rest the ends of the spans on that, in which case the load could be taken from the old pier entirely. A defective pier in the Atchafalaya River Bridge, on the New Orleans, Texas and Mexico Lines, was handled in this way, but probably the plan pursued by Mr. Smith was, in the end, the best for the Arkansas River.

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* St. Louis, Mo.

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DEPRECIATION AS AN ELEMENT FOR CONSIDERATION IN THE APPRAISAL OF PUBLIC SERVICE PROPERTIES.

Discussion.*

By Messrs. F. W. Green, Richard J. McCarty, and Philip W. Henry.

Mr. Green.

F. W. Green,† Assoc. M. Am. Soc. C. E. (by letter).—Of late, one might say that the subject of depreciation has become the very apotheosis of platitudinous piffle, on the part of some writers, of whom it may regretfully be said, *"damnant quod non intelligunt"*. In the present instance, however, the author has handled his subject most commendably, and has shown a thorough knowledge of its fundamentals, which could only have been the result of studious application and long experience. He has covered the subject so thoroughly that very little else need be said.

The recent papers before this Society by William J. Wilgus‡ and John W. Alvord,§ Members, Am. Soc. C. E., and the present paper by Mr. Grunsky, will undoubtedly become milestones in the development of the subject of valuation; and the latter paper happily supplements the two former, both as to logical presentation, and clear enunciation of principles.

It might not be amiss, here, to give the following definitions:

"Value. Specif.: *Economics.* a—Efficiency in exchange; power which an object confers upon its possessor, irrespective of political compulsion or personal sentiment, to command the commodities and services of others; purchasing power in the abstract. b—Concrete

* Continued from January, 1915, *Proceedings.*

† Stamps, Ark.

‡ "Physical Valuation of Railroads," *Transactions,* Am. Soc. C. E., Vol. LXXVII, p. 203.

§ "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return," *Transactions,* Am. Soc. C. E., Vol. LXXVIII, p. 788.

Mr. Green. purchasing power; the specific quantity of another object for which a given object can be exchanged; a price which can be actually obtained. The value of an article depends not upon its *total* utility, but upon its *marginal* utility (See under Utility), diminishing as the supply increases. It will usually be proportionate to the cost of production, because when the value of an article is above its cost producers will tend to increase the supply, while if its value is below its cost producers will tend to diminish the supply. c—Proper price; the quantity of money, goods, or services which an article is likely to command in the long run, as distinct from its price in an individual instance; a legitimate price, as distinct from an unfair or extortionate one;—sometimes called *normal value,* in contrast to *market value.* 'The commercial or competitive theory bases *value* upon what the buyer is willing and able to offer for an article; the socialistic theory bases it upon what the article has cost the seller in the way of toil and sacrifice.'—*A. T. Hadley.* d—The estimate which an individual places upon some of his possessions as compared with others, independently of any intent to sell;—sometimes called *subjective value,* or, less correctly, *value in use,* and employed in a loose sense, as nearly equivalent to *utility. Value* in use is utility and nothing else, and in political economy should be called by that name and no other.—*F. A. Walker.*

"Utility. Power to satisfy human wants. 'Wants are here reckoned quantitatively with regard to their volume and intensity; not according to any ethical or prudential standard' (*A. Marshall*). Modern economists distinguish the *total utility* measured by the loss which would be involved in the entire deprivation of a commodity, from the *marginal utility,* or *final utility,* which would be involved in the loss of a small amount of it. Thus, the *total utility* of water is infinitely great; but the *marginal utility* of a single quart of water is ordinarily very small, because a man or a community has usually so much water at command that one quart more or less makes very little difference.

"Depreciation. Act of depreciating, or state of being depreciated; specif.: a—A falling of value; of money, a reduction or loss in exchange value or purchasing power, esp. with reference to the face value. b—A lowering in estimation; disparagement."

The foregoing are from Webster's New International Dictionary. The following is from the Century Dictionary:

"Depreciation. 1. The act of lessening or bringing down price or value. 2. A fall in value; reduction of worth. 3. A belittling or running down of value or merit; conscious undervaluation or underestimation of the merits of a person, action, or thing; unfavorable judgment or scant praise."

Perhaps some of the confusion extant, as to depreciation, in a valuation sense, has been due to some writers following Definition 3 through misapprehension.

In the 11th edition of the Encyclopedia Brittanica, p. 867 *et seq.,* Vol. XXVII, under "Value", will be found some interesting information; also in Dr. Taussig's "Principles of Economics," Vol. II, pp. 363-418, incl.; both are commended to the diligence of the uninformed.

Mr. Green.

A booklet, entitled "Depreciation of Railroad Property", by Richard J. McCarty, M. Am. Soc. C. E., is a very valuable contribution to the literature of this subject.

It is equally to the interest of the rate-payer and the utility that portion of those who have submitted their views, obviously confound physical depreciation with functional depreciation; hence a few remarks may perhaps be apropos.

It is equally to the interest of the rate-payer and the utility that operating expenses, taxes, and interest, should be at such a minimum as is consistent with economy and good service. Depreciation is legitimately a part of operating expenses, and its sole function is to maintain, unimpaired, the original capital investment. (It would be silly to say that this depreciation fund must be in specie, and conspicuously displaced by the utility in a glass case, to the end that the rate-payers might be well assured, as often as desired, that the utility was actually possessed of the necessary funds to maintain unimpaired service. If the utility could not be freed from the imputation of truculence, it should never have been permitted a corporate birth.) The measure of economy is the requirement that the annual cost of an article shall be a minimum. Annual cost is the sum of interest, maintenence, and depreciation. Hence, the rate-payer profits in every case where the utility follows the principle of minimum annual cost. It is against the interests of the rate-payer, as well as those of the utility, to base rates on depreciated physical value. The classic illustration of the railroad cross-tie will make this clear. Suppose it were possible for a railroad to purchase a tie of such lasting qualities that it would have a serviceable life of 50 years. This tie will function at 100% efficiency for 50 years, and it is to the rate-payer's interest that it serve its functional life before taken out of the track. At the end of 25 years, however, the utility will be entitled to receive, the premises considered, a rate of return based on a 50% physical valuation. There is no incentive, therefore, for the utility to get the full service out of the tie, but, on the contrary, the sooner it is renewed, the higher the value on which rates are presumed to be based. In such a conflict of interest, it is but natural to conclude that the rate-payer will be required to pay rates based on extravagant maintenance, which has been forced on the utility by the rate-payer; the latter, indeed, has deducted what some have euphemistically styled "the return of capital to the owner", but, in so doing, has quite deservedly been beaten at his own game. In such case, the sum of interest, maintenance, and depreciation would hardly be a minimum; hence, the economy which should inure to the benefit of both the utility and the rate-payer is non-existent. It is *in posse,* but, in the case assumed, not *in esse.* The conclusion, therefore, seems inevitable that, if the interests of both the utility and the rate-payer are properly to be conserved, valuation should comprehend functional depreciation, but should exclude physical depreciation.

Mr. McCarty.

RICHARD J. MCCARTY,* M. AM. SOC. C. E. (by letter).—This paper seems to be a long step in the right direction toward clearing the way of many of the obscurities which envelop the question of valuation, and it is hoped that it will be carefully read and given the attention it deserves by all who are interested in that important subject.

Mr. Grunsky denominates his method of making appraisal as the "Unlimited-Life Method", but it would seem that, with equal propriety, it might be called the "Common-Sense Method", particularly with respect to the method of dealing with depreciation.

All theories of valuation which involve the deduction of depreciation from cost of reproduction in the determination of what is known as "fair value" fail to take into consideration certain important practical features involved.

There are three possible bases on which to estimate the cost of reproduction of physical property of a public utility:

1.—The cost, by present methods, and under present conditions, of an equivalent property capable of rendering the same service as the existing property;
2.—The cost, by present methods, and under present conditions, of a property identical with the existing property; and
3.—The cost of the existing property, at present prices, with the methods and conditions under which the property was created, developed, and brought to its existing state.

The theory of cost of reproduction of physical property is based on two principles, each of which is sanctioned by commercial custom, is in accord with accepted theories of economics, and is justified by the Court decisions that will be quoted under each.

The first of these principles is: The investors are entitled to include in "fair value" the proper original cost or all values honestly and judiciously sacrificed in producing the property.

This principle is justified by the Supreme Court of the United States in Reagan *vs.* Farmers Loan and Trust Company, thus:

"And yet justice demands that every one should receive some compensation for the use of his money or property, if it be possible, without prejudice to the rights of others." (154 U. S. Sup. Ct., 362.)

Here, surely, "some compensation for the use of his money or property" cannot be taken as meaning less than a fair rate of return on all money or property actually invested in good faith and with good judgment in a property which is able to justify its existence, and manifestly such a return for such a property cannot prejudice the rights of others.

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* Kansas City, Mo.

Mr. McCarty.

The principle, however, was expressly affirmed by the Court of Appeals of New York, on March 24th, 1914, in People *ex rel.* Kings County Lighting Company *vs.* Willcox *et al.*, as follows:

"The right to limit the corporation to a fair return fixed by public authority necessarily involves the correlative right in the corporation to be assured of that fair return during all the time that its capital is employed in the public service." (N. E. Rep., May 5th, 1914.)

The second principle is: The investors are entitled to any appreciation due to increased prices of land, labor, or materials invested in the property.

This principle is justified by the Supreme Court of the United States in Willcox *vs.* Consolidated Gas Company thus:

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made concerning rates. If the property which legally enters into the consideration of rates has increased in value since it was acquired, the company is entitled to such increase." (212 U. S., 52.)

Here, "the property which legally enters into the consideration of rates", naturally includes all land, labor, and materials invested in the property.

The Cost, by Present Methods, and Under Present Conditions, of an Equivalent Property Capable of Rendering the Same Service as the Existing Property.—An equivalent plant, as a rule, will involve either more or less land, labor, and materials than has been invested in the existing plant. If more, then the investors would be receiving the benefit of physical property they did not invest, which cannot be justified. If less, then the investors would be deprived of a certain portion of the investments actually made, and the first principle set forth would be violated.

Again, under the cost of reproduction of an equivalent plant, the investors either would secure increased prices of land, labor, and materials which had never been invested, which is unfair, or would be subjected to decreased prices of land, labor, and materials which they had actually invested in the property, which is in violation of the second principle.

The Cost, by Present Methods, and Under Present Conditions, of a Property Identical with the Existing Property.—In an identical property the land and raw materials would be practically the same as those involved in the existing property, and the cost of reproduction of these items would be no violation of the principles mentioned, except in so far as the prices would be less than those originally paid, in which case the cost of reproduction would violate the first principle set forth, because it would subject the investors to a deduction from the proper original cost of the property.

Mr. McCarty.

Again, in an identical property, built by present methods, the amount of labor, except by accident, would be either greater or less than in the existing property. If greater, the investors would get the benefit of labor they did not invest, which cannot be justified. If less, they would be deprived of a certain portion of labor they did invest, and the first principle would be violated.

Moreover, should the reproduction labor be greater than the invested labor, the investors might get increased prices of labor not invested, which is unfair. If the reproduction labor should be less than the invested labor, the investors might be denied increased prices on certain labor which they did invest, and the second principle would be violated.

The Cost of the Existing Property, at Present Prices, with the Methods and Conditions Under Which the Property was Created, Developed, and Brought to its Existing State.—This method of estimating the cost of reproduction gives the identical amount of land, labor, and raw materials used in the existing plant at current prices, and complies with both of the principles set forth, except in case the present prices should be less than the prices actually paid in creating and developing the existing property. In that event, the investors would be subjected to a deduction from proper original cost of their property, which is in violation of the first principle set forth.

In view of all these considerations, it seems clear that,

1.—The proper basis for estimating the cost of reproduction is the cost of a property identical with the existing property, at present prices, with the methods and conditions under which the existing property was developed.

2.—In estimating the cost of reproduction, the items should be determined so that those which are greater than the original cost can be separated from those which are less.

3.—In using cost of reproduction for determining "fair value", only those items are fairly available which are equal to or greater than the corresponding items of proper original cost.

Evidently, therefore, the general theory of cost of reproduction which subjects the investors of a public utility to a loss, because of decreased prices of land, labor, and materials, cannot be justified, because it violates the recognized principle that they are entitled to a fair return on their capital during all the time that it is used in the public service.

It will be held, of course, that, if they are to receive the benefit of an increase in prices, they should stand the losses incident to a decrease. This, of course, is true of private investors, who have the right to dispose of their property to the best advantage at any given time and recover their losses by other investments. Investors in

public utilities have no such right. Therefore, they should not be subjected to the consequences of a right that they do not enjoy. Mr. McCarty.

However, even were the cost of reproduction determined by making proper allowance for all decreases in price, the deduction on account of depreciation cannot be justified. For instance:

Suppose a new railroad, from the time it commenced operation, should charge to its operating expenses an amount sufficient to take care of the depreciation which begins in each perishable part at the moment of its installation, and to provide a fund for renewing that part when it shall have been worn out.

In the course of time the property will settle down to a condition at which the depreciation of the whole will be an average of from 10 to 25% of the original cost. This depreciation in no way affects the safety and efficiency of the property, and cannot be made good until the parts reach the limit of their usefulness, without a waste of serviceable material.

In case the original cost should be, for illustration, say, $10 000 000, there would at that time, and in the event the property had been able to earn it, a fund of from $1 000 000 to $2 500 000 collected in the way of rates and set aside to take care of this accrued depreciation; but, as the depreciation cannot be made good without an undue waste of serviceable material, this fund could not be applied to the purpose for which it had been created. This large sum would be paid through rates in excess of those actually necessary, and at a time when the public would be least able to pay, so that the impropriety of such a course would seem to be sufficiently clear.

The only justification for the deduction of this fund from the so-called "fair value" of the property would be its complete and unconditional restoration to the investors, that is, they must be put in precisely the same position with respect to it as before its investment in the railroad property. Therefore, if deducted from "fair value", it must be distributed among the security holders, not as dividends, but as return of capital. This would at once raise the question as to whether the security holders, having legally invested without any such condition, could be compelled to accept such payment, and, if so, whether the bond holders should participate in the distribution, and, if so, on what basis and in what manner could the distribution be made. Furthermore, it is well known that no man can determine the amount of depreciation in question, and that no two men seem able to agree on this point.

It seems clear, therefore, that the conclusion reached by Mr. Grunsky, to the effect that, in the appraisal of a public utility, all considerations of depreciation are unnecessary, is entirely sound.

Mr. Grunsky quotes the decision of the Supreme Court of the United States in Knoxville *vs.* Knoxville Water Company, as follows:

Mr. McCarty.

"If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of the unwarranted dividends upon over-issues of securities, or of omission to exact proper prices for the output, the fault is its own."

As Mr. Grunsky states, the Court in this ruling assumes that the amount of profits of a public utility can be increased or decreased by increasing or decreasing the rates, and fails to recognize that an effective way of destroying the business of a public utility is by an undue increase in rates. The reason that this feature was overlooked by the Court is no doubt due to the fact that it was not properly developed by counsel at the trial of the case. This, in turn, was no doubt due to counsel not having been posted by the management in all the practical details of the situation.

This shows the importance, not only of establishing a correct theory of valuation, but of establishing it in such a way as to be perfectly clear to the legal mind, to the end that it shall be properly presented to the Courts.

Mr. Henry.

PHILIP W. HENRY,* M. AM. SOC. C. E. (by letter).—Not always do engineers and accountants agree in matters relating to the keeping of accounts, but, in regard to the definition of depreciation, they are in very close accord. For instance, in Keister's "Corporation Accounting and Auditing" depreciation is defined as: "The actual loss upon assets which are diminishing in value or it is an estimated sum charged against gross revenue; which amount is considered sufficient to replace the capital used up or reduced by wear and tear"; and Bentley's "Science of Accounts" states that "Depreciation of a fixed asset indicates its probable decrease in value by reason of wear and tear, and the efflux of time".

For the definition of depreciation by engineers, we have that given by the author as "the lessening in worth of any perishable article or property." Annual depreciation is the "annual theoretical lessening in worth, expressed in dollars". Another definition is that given by Mr. Alvord† in his paper on "Fundamental Principles of Public Utility Valuation": "The lessened value of any property, structure, or machine, due either to its wear, loss of usefulness, growing lack of adaptation or approaching abandonment".

The definition, however, which, on account of its authoritative source, is controlling to individuals and to corporations of all kinds—public utility or otherwise—is that given by the Internal Revenue Bureau of the Treasury Department in connection with the income tax. It is interesting to note how close the agreement is between the definition given by the accountants and engineers already quoted and

* New York City.

† *Proceedings*, Am. Soc. C. E., for October, 1914.

that of the Internal Revenue Bureau. On Form 1035, "Return of Annual Net Income", for Miscellaneous Corporations, Section 2, approved by Congress, on October 2d, 1913, a deduction from gross income is allowed for "Total Amount of Depreciation". This item is defined more particularly: "The amount claimed under Item 5 (for depreciation) should be such an amount as measures the loss which the corporation actually sustains during the year in the value of buildings, machinery and such other property as is subject to depreciation on account of wear and tear, exhaustion or obsolesence". Mr. Henry.

It is evident, therefore, that the confusion which exists in regard to depreciation is caused, not so much by disagreement on what depreciation really is, as to the manner of applying it to any given case. Therefore, although the principles governing depreciation are the same, whether applied to a mining property having a definite life time, or to a public utility corporation having an unlimited life time, because certain physical assets of both are subject to "probable decrease in value by reason of wear and tear, and the efflux of time", the method of applying these principles must necessarily be different. With both types of corporations, however, it is necessary to set up a depreciation reserve, and, in both cases, the amount in the depreciation reserve at any given time should equal original value less present value, using cost and value as equivalent terms and neglecting other elements (than depreciation) affecting valuation. The author brings out this principle when he states that "The investment less this obligation to replace (also less deferred maintenance) is, in other words, a measure of present worth".

From the definitions given by the author, it is evident that investment and original value are equivalent terms, and that obligation to replace plus deferred maintenance is nothing more than depreciation reserve. It seems unfortunate that the author should have used these terms instead of depreciation reserve which has an accepted meaning among engineers and accountants, and is recognized by the Internal Revenue Bureau, as follows: "Where a depreciation reserve is set up, all renewals and replacements must be charged to such reserve, and the addition to this reserve each year must be a fair measure of the loss which the corporation sustains by reason of the depreciation of its property." Also "Where depreciation of physical property is made good by renewals, replacements, repairs, etc., and the expense of such renewals, replacements, repairs, etc., is charged to the general expense account, no deduction for depreciation can be made in the Return of Annual Net Income." This ruling indicates that depreciation reserve should be set up at the beginning of operation, and that it should truly represent "the loss which the corporation sustains by reason of the depreciation of its property."

Mr. Henry.

In the principle of present value plus depreciation reserve equaling original value, it is assumed that original value and original cost are equivalent terms, whereas, in practice, it is known that the original cost may be considerably greater than original value owing to bad judgment or bad management; or, on the other hand, original cost may be less than original value through the purchase of a property at a forced sale.

It is evident also, as a principle, that although the money in the depreciation reserve may be invested either in outside securities or in the property itself for betterments and additions, it should not be available for the payment of dividends, but only for making replacements or for the repayment of capital.

It is also evident that the amount in the depreciation reserve will not necessarily increase each year, for because of the large replacements being made in a given year, the amount in the depreciation reserve at the end of that year may be less than at the beginning. Present value, however, will be increased by the exact amount that the depreciation reserve is decreased.

With a depreciation reserve carried in accordance with the principles laid down, no difficulty arises as to whether a public utility corporation should receive the return authorized by some public service commission on its original value or on its remaining value, for, at all times, the property is kept up to its original value through the amount in the depreciation reserve. In buying such a property, the purchaser will be quite willing to pay the original value (cost), for the original value will be there in present value plus depreciation reserve. In buying a corporation which has not set aside a depreciation reserve, the purchaser will naturally pay only the amount represented by present value, but, at the same time, he must assume an obligation to put into the property, later, an amount equal to that which the corporation should have carried in a properly adjusted depreciation reserve.

As to the method of setting aside depreciation reserve and its treatment on the books, the writer has already given his views in his discussion on "Valuation for Rate-Making Purposes."*

* *Proceedings*, Am. Soc. C. E., for April, 1914.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE WATER-PROOFING OF SOLID STEEL-FLOOR RAILROAD BRIDGES

Discussion.*

By Messrs. Henry H. Quimby, Ralph J. Lawrence, A. T. Goldbeck, F. Auryansen, J. H. Fichthorn, Edward W. De Knight, and Albert H. Rhett.

Mr. Quimby.

Henry H. Quimby,† M. Am. Soc. C. E.—Has the author ever tried coal-tar pitch for water-proofing? If not, why not? Also, why is asphalt mastic generally regarded as an undesirable material, as stated in the paper? Also, why does the author call the drainage of the bridge seat a bad detail, when the only examples of it which he mentions have given no trouble? Generally the bridge seat is exposed more or less to rain, and, therefore, water will reach it independently of any drainage from the bridge floor; consequently, it is good practice to drain the bridge seat back from the face, even when the floor does not discharge on it. This has been the speaker's practice, and it has proved satisfactory. It ought to be possible to secure such drainage anywhere without soiling the face of the abutment, and only a direct connection of the floor drains to a sewer, which is not always available, will be less subject to freezing and overflows in cold weather.

One of the written discussions seems to advocate the use of cement mortar for sealing against the webs and girders. That indicates a great deal of faith in the adhesiveness of cement mortar to steel. Observation of a number of cases shows that the tendency of cement concrete is to leave the surface of the web of a girder, no matter how elaborately it has been made fast to it by embedded steel anchors. This is probably due quite as much to the expansion and contraction of the surface from moisture as it is to vibration, for it has ap-

* Continued from January, 1915, *Proceedings*.

† Philadelphia, Pa.

Mr. Quimby.

peared on structures which are subject to very little vibration. The surface of the concrete, absorbing water, expands against the steel and is probably compressed—permanently compressed—and when it dries out, it pulls away from the surface of the steel, leaving an opening which admits water. An adhesive and plastic substance, like that described in the paper, in a groove from which it cannot flow, seems to be necessary for water-tightness. A sloping hood over this is in the nature of counter flashing, and to the extent that it sheds water from the joint below it, it will certainly reduce the danger of leakage.

The suspended gutter under the bridge floor has its objections—it becomes completely dammed up with dirt, and, unless made very stout, will be dented and bent up. Also, generally, it must be either small and of flat grade or it compels a reduction of the floor depth by trenching on the clearance. Therefore, the most satisfactory design should be the shedding of the floor drainage over the back wall, and also draining the bridge seat through the back wall. Ample capacity for vertical drainage back of the abutment can be afforded by large pipes either to a porous stratum of gravel, if one is there, or to a sewer if one is available, or, failing both these, through the sidewalk to the street gutter. All these can be made accessible for cleaning out obstructions, and where a direct connection to a sewer is not available any drain is likely to freeze up. In freezing weather, the type having a flow over the back wall will be least interfered with.

Mr. Lawrence.

RALPH J. LAWRENCE,* ASSOC. M. AM. SOC. C. E. (by letter).—Although the object of this paper is to submit a specification for the design, materials, and manner of application of the water-proofing of railroad bridges with solid steel floors, and to call attention to the necessary details that should be considered by the designer when engaged on such structures, it would be well to know, in view of the author's experience with work of this class, just what method or kind of water-proofing he would suggest for the different types of bridges, water-tightness, durability, and cost being considered.

The writer wishes to refer particularly to the latter part of the Synopsis, where the use of felt, fabric, or asphalt mastic is discussed. As the author does not mention coal-tar pitch, he is evidently of the opinion that the floor of a steel railroad bridge is no place for such material, and, if such is the case, the writer agrees with him.

As to the use of felt, fabric, or asphalt mastic, the writer desires to know which of these materials the author thinks the proper one to use in designing a water-proofed structure, all conditions being equal.

The paragraph referred to states that, if hard pressed for room, asphalt mastic is indicated, thus leaving one under the impression

* Philadelphia, Pa.

that felt or fabric is more satisfactory than the mastic, and that this material should only be used where necessity demands it. Mr. Lawrence.

The felt or fabric, with the brick protection, takes up approximately 2 in. more room than the mastic, and, if they are considered superior to the latter, it would appear to the writer that this slight difference in thickness should not compel its use. The concrete filling required on most bridges must be given a superficial grade on account of drainage requirements, and it would be better to do with a little less ballast at the summits than use an inferior water-proofing material.

It is admitted that felts and fabrics are subject to disintegration, and that a properly prepared mastic is more durable than either of them; then why not use mastic, not only where there is barely enough room to get it in, but where there is sufficient clearance to apply the details used in water-proofing with felt, as shown in Figs. 4 to 9, inclusive.

It is believed that a properly prepared asphalt mastic, 1½ in. thick, is more water-proof and durable than a few films of asphalt placed between several layers of felt or fabric, provided cracks, caused by contraction and the deflection of the steelwork, can be avoided.

Although quite a number of bridges designed by the writer under the author's supervision are water-proofed with felt, and are tight and giving satisfaction, he is of the opinion that a mastic properly prepared with low-melting-point asphalts, properly placed and protected, will prove more effective, more durable, and cost less than the felts or fabrics. The mastic should be placed on the concrete base, and not directly on the steel. It should be laid in blocks, 1 or 2 in. apart, the joints being located at stiffeners, gussets, at the ends of column-bearing girders, etc., and filled with a ductile, adhesive asphalt, having a low melting point such as that specified for the web and stiffener joints. The mastic should then be protected with about 2 in. of granolithic concrete, reinforced with welded wire cloth, to exclude dirt and retain the asphalt during hot weather.

It is hoped that the author will supply additional information on the subject he has so ably presented for consideration.

A. T. Goldbeck,* Assoc. M. Am. Soc. C. E. (by letter).—As has been indicated by Mr. Wagner, one of the very important properties to be possessed by a water-proofing material is that of adhesiveness, and this is of special importance in materials for railroad bridges, as the excessive vibration in these structures tends to loosen the adhesive bond. It is realized that different kinds of water-proofing compounds vary in this property of adhesiveness, depending on their characteristics and, obviously, on the surface to be treated. Mr. Goldbeck.

For the purpose of obtaining quantitative information on the relative degree of adhesion of various kinds of water-proofing materials

* Philadelphia, Pa.

Mr. Goldbeck.

to the surfaces of different characters, the writer has conducted a series of tests, the results of which may be of interest at this time. These tests were undertaken in connection with the water-proofing studies of the author and Mr. James W. Phillips. Before the adhesion of bituminous materials could be obtained, it was necessary to develop an adhesion test, and the scheme finally adopted, although not entirely satisfactory, seemed most nearly to conform to practical conditions. The method of testing the adhesion was as follows:

Specimens, whether of steel or concrete, were prepared, 3 in. in width. A strip of cotton duck (Ottawa sand bagging), 3 in. wide, was immersed in the bitumen to be tested when liquefied at a temperature of about 300° Fahr. The treated duck strip was laid on the steel or concrete specimen, while still hot, and rolled with a steel roller under moderate pressure until cool. The concrete specimens were mixed in the proportion of 1:2:4, and were about 1 week old when the tests were made. In each case the cut-back treatment was applied at least a few hours before the bitumen was to be tested, and with the concrete surfaces thoroughly dry and clean. One day after the preparation of the specimens, they were placed in the tank of a ductility machine containing water at 40° Fahr. After an immersion of about 1 hour, the treated duck strip was peeled off at the rate of 5 cm. per min., and the load was read on a spring balance attached to the moving carriage and to one end of the strip. The spring balance was parallel to the specimen, and, as the carriage was moved along, the duck strip was peeled gradually (not slid) from the surface of the specimen. In general, the load recorded by the spring balance remained fairly uniform throughout each particular test, and the average load was taken as representing the adhesion. The results of the test are given in Table 4.

These results are very instructive, and point out quantitatively the adhesive qualities of different types of materials to various kinds of surfaces. Note how much better the adhesion is with steel than with concrete. With few exceptions, this seems to be a general rule, and shows that where the bitumen comes in contact with two surfaces, one of steel and the other of concrete, particular care must be taken to obtain good adhesion to the concrete. Such a condition arises in the pocket of bitumen, called for in Paragraph 41 of Mr. Wagner's specifications, adjacent to the steel girders. Note the almost entire lack of adhesion of the bitumen to the untreated surface of concrete, shown in Column 10 of Table 4. The failure here was really a concrete failure, due to the stripping off of the non-cementitious layer of laitance from the troweled surface. A cut-back treatment applied to both steel and concrete in general aids the adhesion, especially in the case of the concrete. In Column 13 of Table 4 (concrete specimens) are given the results obtained with a surface freed from the top

Mr. Goldbeck.

TABLE 4.—Adhesion Tests of Water-Proofing Bitumens.

Compounds.	Load Required to Strip Off Duck Strip, 3 In. Wide. Temperature, 40° Fahr.															Ductility, in Centimeters, at :			Penetration.			Analyses.				
	Adhesion to Steel Plates, in Pounds.								Adhesion to concrete, in Pounds.																	
	Mill scale not removed.				Scraped and wire-brushed.				Troweled surface.				Surface next to form.													
	Untreated.	Painted with cut-back.	Painted with red lead, then cut-back.	Painted with red lead.	Untreated.	Painted with cut-back.	Painted with red lead, then cut-back.	Painted with red lead.	Untreated.	Painted with cut-back.	Acid treatment.	Acid treatment plus cut-back.	Untreated.	Painted with cut-back.	Acid treatment.	Acid treatment plus cut-back.	40° Fahr.	77° Fahr.	110° Fahr.	40° Fahr.–2009–1 min.	77° Fahr.–1009–5 sec.	At 77° after 5 hours heating at 325° Fahr.	Loss on heating for 5 hours at 325° Fahr.	Flow point.	Melting point.	Soluble in CS_2.	Percentage of total bitumen soluble in 88° P. E.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
A.	16	24	32	24–34	18	24	28	26	0	1¾	½	3½	¼	2¾	1¾	7¾	4¾	11½	32	34	47	29	1.28	140°	167°	99.94	69.63
B.	14	18	20	18	8	12	14	9	0	¾	0	2½	½	3½	1¾	6	2½	3¼	41¼	54	54	38	1.70	161°	197°	99.96	69.26
C.	24	32	34	34	24	48	50	34	0	1¾	¾	20	15	30	20	30	6¾	62	85	26	44	28	0.90	125°	161°	99.90	70.54
D.	10	20	24	17	16	20	20	16	0	1	0	4½	0	9	1	11	1¼	3¾	5½	21	24	21	1.79	203°	254°	99.93	59.68
E.	30	34	38	32	26	30	32	20	0	1½	¼	3½	3	12	3	14	3½	11	16½	31	34	19	2.06	149°	179°	77.75	71.27
G.	36	46	50+	35	28	44	50	44	1	5½	1	3	4	35	20	24	6¼	70	74	34	70	34	1.34	122°	153°	99.95	72.22
H.	22	26	30	28	20	22	24	20	½	3	1¼	5	¼	4	¼	8	3¼	4¾	7	28	38	22	4.10	153°	199°	99.94	61.78
J.	26	30	28	28	26	28	32	32	¼	1½	½	5½	½	12	0	10	3½	6½	11	25	31	16	3.69	160°	192°	99.95	62.55
L.	24	20	24	20	24	18	20	16	0	1½	1¼	4	3	16	5	12	4½	10	25½	46	63	35	0.85	149°	170°	99.00	71.63
M.	24	20	24	24	22	24	20	20	0	1¼	½	6	¾	16	2	14	1½	7	16	18	27	18	1.25	154°	190°	99.90	59.76

Mr. Goldbeck.

layer of laitance by using dilute hydrochloric acid, and then painted with the bitumen under test cut back with gasoline. Note the large increase in adhesion to the concrete resulting from this treatment. These results should be of interest to the builder of concrete roads to which a bituminous surface treatment is to be applied. They indicate the absolute necessity of ridding the surface of the non-cementitious layer on the concrete before the application of the bitumen. In Columns 14 to 17, inclusive, are given the results obtained on the concrete surface adjacent to the wooden form. They are generally higher than those obtained with a troweled surface.

Referring to the results of adhesion to the steel surfaces, it will be noted that there seems to be relatively good adhesion irrespective of the character of the surface, whether scraped, unscraped, or painted with red lead. A cut-back treatment, however, seems to be quite generally beneficial. On the steel plates used as specimens, there was very little loose mill scale and no rust. It is well to rid the surface of all loose scale and rust before applying the bitumen.

Regarding the ordinary physical characteristics of the bitumens used, correlated with their adhesive properties, it would seem that ductility and adhesion go more or less hand in hand. Thus, compounds having high ductility seem to have good adhesive properties. It seems well, therefore, to specify a fairly high ductility in order to insure more nearly a material of good adhesion. The value, 20 cm., specified by Mr. Wagner, would seem to insure a material that will adhere well to both steel and concrete surfaces when these surfaces are properly treated. Perhaps it would be well to specify a definite ductility at 40° Fahr., as this temperature approaches more nearly the dangerous one at which the adhesive bond is likely to be broken. As the hardness of the material likewise influences the adhesion, it would be advisable to specify definite penetration limits.

Mr. Aryansen.

F. Auryansen,* M. Am. Soc. C. E.—About ten years ago, when extensive grade-crossing elimination on Long Island was under consideration, among the first problems to be solved was the kind of bridge floor. Although much more expensive, the solid floor was adopted, because, though it would allow continuous ballasted tracks, it would also prevent grease, ashes, coal, or other materials from falling to the street below. Of the three kinds of solid floor—timber, steel, or a combination of reinforced concrete and steel—the latter was adopted as promising to be the most permanent and satisfactory, provided it could be protected from water.

One of the most important reasons for water-proofing bridges is the reduction of danger from electrolysis. In the future, as in the past, the majority of water-proofed bridge floors will unquestionably be within

* Jamaica, N. Y.

Mr. Auryansen.

or near towns and cities, where stray currents from electric light, power, and traction systems are to be expected. Furthermore, electric traction for suburban service is in so much favor in large centers of population, that its extension to through business is, in a number of cases, only a question of time; and although, with the further development of electric utilities, adequate means may be devised to reduce or eliminate stray currents, in the present state of the art, the floors of bridges now built should be protected from this insidious danger.

Wet concrete is an electrolyte, and, at the other extreme, dry concrete is a poor conductor; hence the reinforced concrete floors, not only of railroad, but of highway, bridges, should be kept dry, that is, water-proofed; and it should be the aim in all designs to simplify the problem of water-proofing, so that the desired end may be reached, namely, a tight floor.

Expedients to secure this result are, in order of desirability:

1.—Such grades that the water will run off quickly;
2.—Drainage at back walls;
3.—Water-proofing, either by the integral, mastic, or membrane method;
4.—As a last and unavoidable resort, drain pipes.

Good Grades.—A top of floor grade of 0.7% parallel to the tracks is the minimum slope used. When the track grade is less than this, a summit is provided, and the drainage is carried thence to both ends of the span on 0.7% grades.

Drainage at Back Walls.—For several years, it has been the practice on the speaker's lines to place a course of loose rubble against the back of the abutments for the full width of the bridge floor, and from the original ground level up to the floor, effectually disposing of the wash from the bridge and eliminating the unsightly leakage often found along the horizontal joint between the slab and abutment.

If the water can be simply conducted through the floor and allowed to fall freely to the ground, as in the case of a viaduct on a private right of way, no serious objection can be urged, although in cold weather there may be danger to trackmen or others beneath from falling icicles. Gutters over streets, however, are a nuisance; birds' nests clog them, and high loads on passing vehicles injure and are damaged by them. Pipes, leading from the bridge floors to the street or sewer, freeze and burst, and in the vicinity of New York, for several years, they have been avoided.

About 1909, a few through, girder bridges were built with no deck water-proofing, although flashing angles were provided on the web. Care was taken in the concrete work to secure a dense mixture, and the surface was carefully troweled. Although there is some leakage,

Mr. Auryansen.

no rust has appeared; but no more floors are built without water-proofing.

No faith is placed in the "integral" method of water-proofing, as it cannot cope with slab cracks, construction joints, or the shrinkage of the concrete from the steel. The vibration caused by passing trains also inevitably shakes or jars the concrete while setting, thus inducing very narrow cracks, especially along the under side of horizontal or inclined surfaces of the steel, and at such cracks, capillary action draws in enough water to cause continued dripping below for days after a storm.

The mastic method is fully treated by the author. The only example of this type with which the speaker has had experience, is for a very shallow trough floor, 11 in. from base of rail to clearance, built in 1903. The ties rested on stirrups in the troughs, and the space around them was filled with a soft asphalt mastic. Rail creeping has forced a little of the mastic out, but the remainder is still in good condition.

The membrane method is the principal one used on the Long Island bridges. Five-ply felt served with coal-tar, with the middle layer reinforced, has been used most extensively, and with very good results; burlap with asphalt pitch was used on one bridge; and a number have been protected with a single-ply cotton fabric dipped in special pitch, and after being laid on the freshly coated concrete, the laps and edges are sealed with a hot iron. Recently, a few floors have been treated with a two-ply cotton fabric impregnated with asphalt pitch. The five-ply tar membrane is very satisfactory on level or slight grades, but on inclined surfaces, the tar has a tendency to settle away from the top edge of the membrane.

On deck structures, with flat surfaces and no corners or projections, the conditions are ideal for good workmanship. Perhaps the most difficult points to make tight are where the edges are flashed up on parapets. Until within a year or so, a 3-in. lip or coping was cast in the parapet wall and the membrane was carried up and sealed under the coping. A later modification is the insertion of a copper flashing strip in the wet coping concrete, the projecting portion being bent down over the edge of the finished membrane. On another recent job the membrane, shown by the dotted line in Fig. 10, was carried over the top of the unfinished parapet, on which the coping was then built. In either case, it is important to follow promptly with the protection course of brick or concrete, in order to prevent the sagging of the membrane and the unsealing of the edge.

It is more troublesome and expensive to make through bridges tight, but it is again mainly a question of sealing the edges of the membrane properly. Engineers seeking tight floors must disagree emphatically with the fabricator's suggestion that the flashing angles be discarded, although, at the same time, sympathizing with him as

to their cost. Angles of ¼-in. metal are too thin and will deform too easily; ⅜-in. should be the minimum. The intricate fitting at gussets can be eliminated to a great extent by making the longitudinal-web flashing angle continuous between web splices; the gussets may be cut to permit this, and the small flashing pieces attached thereto at

Mr. Auryansen.

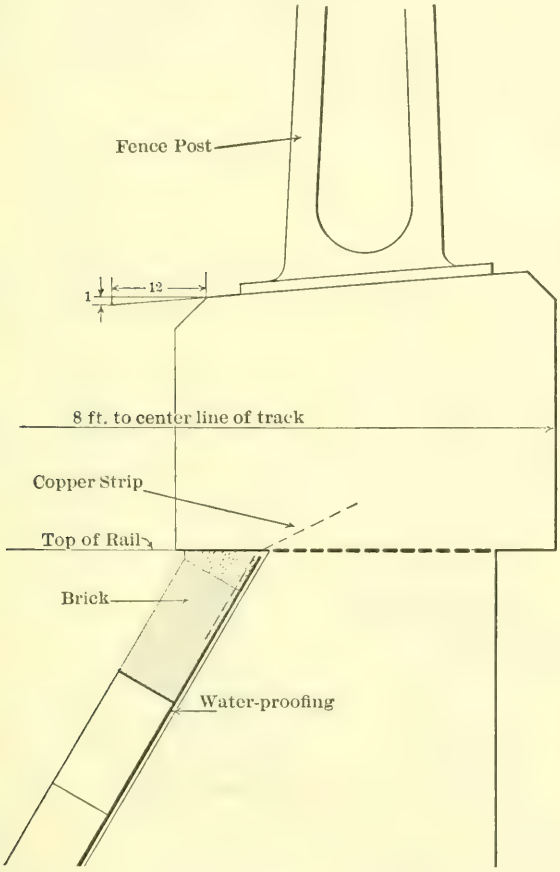

FIG. 10.

a higher plane, as shown in Fig. 11. Fillers should be wider than the flash-angle leg and project above it. The edge of the gusset may be protected by small flaring steel castings or by a cover-plate bent outward at the bottom to flash the membrane and brick. Reasonable care at the shop in fitting these details will make it easily possible

Mr. Auryansen.

for the painters in the field to caulk completely the few small cracks, either with paint skins or red lead putty. A bridge now at the shop is to be fitted with copper strips under the flash angles; these strips are not punched for the rivets, but simply inserted and held in place by the grip of the latter. On completing the membrane and its protection, a trowel-cut should be made in the mortar under the flash-angle to make a dripping edge, and the mortar slot should be painted.

Where clearance will allow, the top flanges of shallow, through girders may be completely encased, which simplifies the problem of

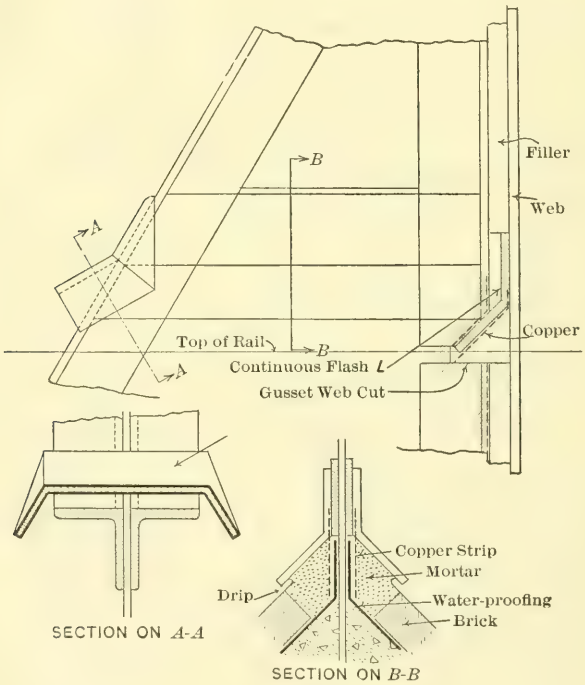

FIG. 11.

protecting the top edge of the water-proofing membrane. When this cannot be done, a continuous flashing angle is attached to the lower edge of the top flange and the membrane is carried up behind it, as shown in Fig. 12.

Other troublesome features are the provisions for contraflexure and expansion. Where the former is anticipated, either the slab and membrane should have additional reinforcement, or a flexible joint should be provided. If possible, expansion should be allowed only at

Mr. Auryan sen.

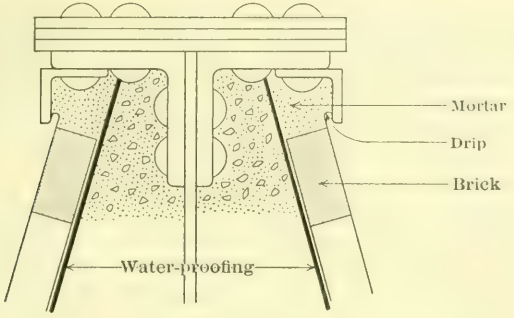

FIG. 12.

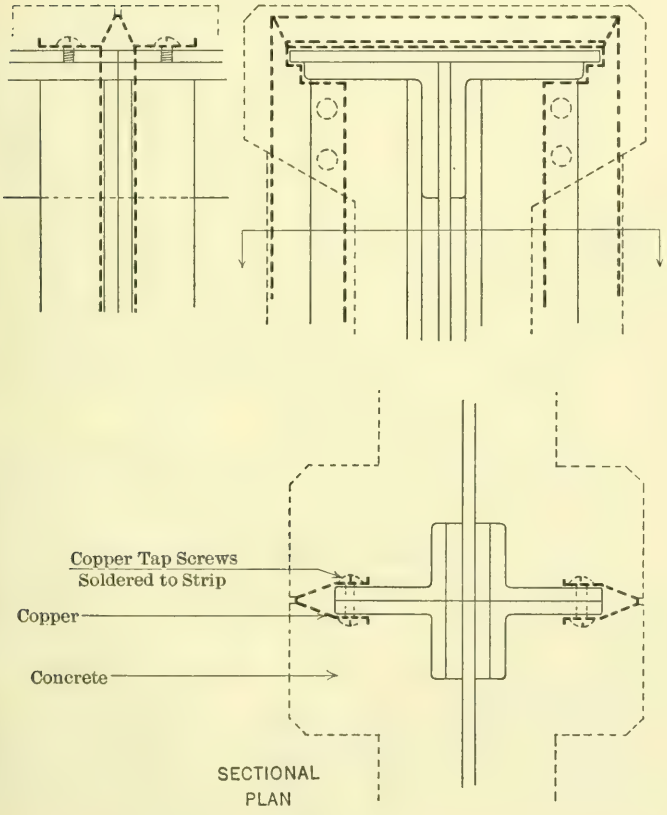

FIG. 13.

Mr. Auryansen.

the crown or summit of the floor. The flat surfaces at such points are not very troublesome, but the parapets of deck slabs and the upper part of completely encased girders should be designed carefully. Concrete parapets and covered girders on recent work have U- or V-shaped copper strips at the top flange joint, and vertically at abutting end stiffeners, as shown in Fig. 13.

Second only to proper design, is the question of workmanship. The utmost care is justified, because of the difficulty and expense of finding and repairing leaks. Work should be done preferably in hot weather and on thoroughly dry surfaces. It is suggested that an asphalt pavement heater be used to warm the concrete to a temperature of from 300° to 350°, which will expel, not only any surface moisture, but much of the air in the pores of the concrete; pitch applied to this hot surface will be kept liquid until forced into the pores by atmospheric pressure. This method has been tried, using a large gasoline torch, and the result was a very intimate bond between the membrane and the concrete. On level or slightly sloping surfaces, this refinement is probably not justified, but on steep or vertical surfaces, the best possible bond is desirable, otherwise the membrane will sag; moreover, with the surface pores completely filled, an additional barrier is provided against leakage through the slab.

Mr. Fichthorn.

J. H. FICHTHORN,* ESQ.—The speaker is very much interested in this excellent paper, principally from the standpoint of the design of the structural steel. The author states: "It is believed that riveting a flashing angle to the web does not provide a satisfactory detail, as it is impossible to make it continuous on account of stiffeners and gussets". The speaker heartily agrees with Mr. Wagner on this point, and is sure that if these flashing angles can be eliminated, the steel fabricator will be everlastingly grateful.

The speaker has recently gone through the experience of fabricating two bridges designed with flashing angles. The first, a double-track bridge of 100 tons total weight, provided with flashing angles ¾ in. thick, which, including the fittings around the brackets, weighed 900 lb. The detailed drawings for this 900 lb. of flashing cost $128, and the fabrication cost $300, making a total of $428 for 900 lb. of material, or $4.28 per ton, on the total weight of the bridge, due to flashing alone. Special care was taken to make a good fit, but the results showed that it was practically impossible to make a tight job around the brackets.

On the second bridge, a five-track structure of 300 tons total weight, a new method was adopted for the purpose of obtaining a better fit and reducing the cost. No detailed drawings were made. The work came from the shop completed, except for the flashing. A draftsman

* Chf. Engr., Lewis F. Shoemaker & Co., Pottstown, Pa.

took cardboard (which material is nearly always used for templets of small details), and fitted and cut and notched until the resulting templet showed the exact contour of the material around which the flashing was to fit. In order to permit of greater accuracy in cutting and bending the material, ⅛-in. metal was used, and, as before, a special effort was made to obtain a tight job. The result was a cost of about $600, or $2 per ton on the total weight of the bridge, without very much improvement over the first case as far as a tight job was concerned. Mr. Fichthorn.

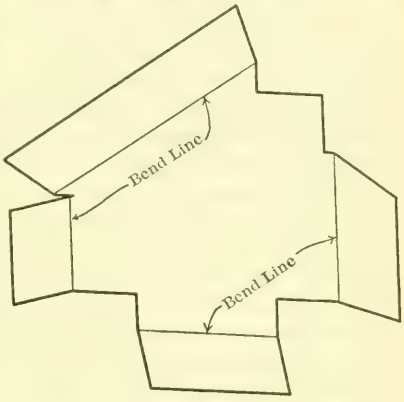

FIG. 14.

The use of malleable-iron fittings around the brackets has been suggested, but the speaker has found that owing to inaccuracies in casting, the results are very little better than, if as good as, steel-plate flashing.

An illustration will probably show the difficulties of fabrication much better than description. Fig. 14 shows one of the simpler templets involved. The reentrant cuts, slits, and bends should be noted. As the thickness of metal is increased, the work becomes more costly and less accurate.

The use of flashing angles increases the cost of the steel very much and lengthens the time of delivery without accomplishing the desired results, and, in the speaker's opinion, this money might well be expended on water-proofing materials of better quality.

EDWARD W. DE KNIGHT,* ESQ.—Failures in water-proofing on bridge floors are due primarily to two causes: First, wrong design of the structure; and second, imperfect application, rather than the kind of materials used. Mr. De Knight.

Imperfect application usually results from two causes: First, the use of labor not skilled in that special class of work, and second, lack of sufficient time and facilities by the railroad for the proper execution of the work.

In the past fifteen years or more, the speaker has examined several thousand bridges in the United States, as well as abroad, and in not one in ten has the floor itself been found to be properly graded. How, otherwise, can the water be drained off? The speaker has never been able to understand why a bridge designer considers in such a peculiar, abstract, almost supernatural, light, a bridge floor to be water-proofed.

* President, The Hydrex Felt and Engineering Company, New York City.

Mr. De Knight.

It is nothing more or less than a roof. Yet a bridge engineer will invariably use, on a bridge floor or deck, a design and materials which he would not dare to use on the roof of a building.

A roof has flashing, it has outlets, and it has a slope. Any consideration, therefore, of water-proof bridge floors must begin, first, with the design of the bridge itself. This design, to be correct (in the water-proofing sense), rests on three essentials: First, flashing; second, outlets; and third, slope.

The flashing must be of the proper kind, and it must be independent of and apart from the water-proofing, precisely as is the flashing on the roof of a building. It is as impossible to do without this flashing in one case as in the other, yet engineers attempt to water-proof bridge floors without proper flashings.

The outlets must be sufficient in number and correctly constructed and in the proper places. Do not make the bridge floor dead flat, so as to hold the water. Slope or grade the floor so as to shed or drain off the water, as on a roof, which is never flat. One cannot fight water; one must lead it, or facilitate its continuous flow to drains and outlets so as to dispose of it, without accumulation, as rapidly as it falls. After all, water-proofing is simply a part of the general scheme of drainage.

As to materials, something has been said about mastic. One would not place on the roof of a building a mastic, like an asphalt pavement, in order to make it water-tight, because it is known that a mastic or asphalt pavement will crack in time and let the water through the cracks to ruin the interior of the building. Therefore, one is afraid to use or depend on mastic. Yet an engineer would use mastic on an arch or a bridge floor to make it perhaps water-tight, when he well knows that it is not dependable for the roof of a building and that it is neither good practice nor roofing practice to use it thus. The speaker repeats, that he cannot understand why a bridge designer considers so peculiarly and abstractly an arch or bridge floor to be water-proofed, as compared to an ordinary, every-day roof.

One word as to the protection of the water-proofing. The speaker's experience is that hard-burned, vitrified brick, laid and grouted in cement mortar, is the best protection for the water-proofing membrane, and is superior to mastic: First, because the brick can be readily removed in sections, and the water-proofing examined and repaired, if necessary; whereas mastic cannot be removed so readily. Second, mastic also has the serious disadvantage that at times the ballast and ties will sink into it, especially in hot weather, and form pockets to hold water and thus hasten decay. This has proved to be so repeatedly. If the mastic is made soft enough not to crack in winter, it will be too soft in summer, and if made hard enough not to soften too much in summer, it will crack in winter. A bridge floor exposed to extremes

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of weather and subject to heavy traffic, presents conditions quite different from the floor of a freight or warehouse. It was because of the failure of mastic years ago, on arches, viaducts, and similar work, that several roads, as, for instance, the Pennsylvania Railroad, discontinued its use, especially on arches and bridge floors, and adopted the more yielding and dependable membrane method. The use of mastic, therefore, is retrogressive and not progressive.

Under materials, Mr. Wagner refers to felt, burlap, cloth, and asbestos, and mentions a trial of one or two layers of cloth—possibly two—on a bridge floor. Now, it is a mistake to use on a bridge floor, subject to the impact of heavy trains and to almost constant vibration, fewer than four layers of any material, no matter whether it be cloth, felt, or burlap; and it would be far better to use five or even six layers. Five is the general standard. The speaker assumes that cloth is specified in only two layers because of its great expense.

The speaker discusses all these materials without bias because his Company manufactures treated cloth, treated felt, treated burlap, etc., and from a manufacturer's standpoint, it does not matter whether one or the other is sold. From a technical, engineering standpoint, however, and from a long and varied practical experience in the sale, use, and application of the materials, he can recommend only those materials which he candidly believes are best suited for the special purposes and conditions. He would not for a moment, therefore, recommend the use, for water-proofing bridge floors, of only two layers of any one of the previously mentioned materials, whether felt, cloth, or burlap. Two layers of cloth, for example, in place, cost as much as five layers of a good, tough, wool-mixed felt. These five layers form a membrane or stratum of water-proofing approximately $\frac{3}{4}$ in. in thickness, whereas two layers of cloth, or felt, or burlap, form a stratum about only $\frac{1}{8}$ in. thick. It requires no argument to prove that, all things being equal, a stratum of water-proofing about $\frac{3}{4}$ in. thick will be more durable than one only $\frac{1}{8}$ in. in thickness. If the stratum is not durable, of what value is it? It is the material in a mass which insures its durability. The additional strength in the thin stratum, because it contains cloth, does not add one iota to this durability, because strength cannot make up for mass and durability. Therefore, if one wishes to use cloth, or burlap, or felt, to obtain the durability which is insured by the mass, one must use not fewer than four layers, or better, as previously suggested, five or six layers. However, as four and five layers of the right kind of felt have given perfect service for all practical purposes for many years, yielding in the mass to all the natural contraction, expansion, and settlement of concrete, and vibration on bridge floors, it is unnecessary to go to the expense of using four or five layers of cloth costing more than double that of five layers of felt, because this

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means an enormous increase in cost for a strength in the cloth, which is necessary only in imagination and not in fact. The factor of safety, in this case, does not require 100% increase in cost. Strength in water-proofing will not hold concrete together or prevent it from cracking. Abundant experience and practice show that four or five layers of a good felt, forming a water-proof mass approximately $\frac{3}{4}$ in. in thickness, in practically every respect and everywhere, has served satisfactorily the mobility of concrete, and at half the cost of cloth, the strength of which is not necessary. There is no real need, therefore, for incurring double expense for an unnecessary strength.

Recently the speaker inquired of an engineer who had designed a large power-house, "What do you purpose specifying for the foundation water-proofing"? He mentioned two layers of fabric. The speaker inquired what he intended to use on the roof. He answered, following the usual custom: "Five layers of felt". The speaker asked him if he had not the principle reversed. Why not invert the structure, and place the five layers below grade and the two layers on the roof? When asked if he would specify two layers of any felt or fabric for a roof, he said: "No, because I do not think it would endure". The speaker replied: "If two layers would not serve on the roof, where you have simply the elements to contend with, and you can always reach and inspect the material, by what logic do you assume that two layers will endure below grade, where it will always be subject to water, the salts in the earth, alkali in the concrete, etc., and where you can never see or get at it? This being so, would it not be better to use, below grade, twice instead of one-half the quantity of materials, that is, if five on the roof, why not ten below grade? This would seem the logical procedure". He remarked that this phase of the problem of water-proofing had never previously occurred to him; that if the mind of an engineer could be brought from other things to think seriously of the technical side of water-proofing in such a light, it would do an immense amount of good.

In considering whether to use felt, burlap, cloth, or asbestos, we again revert to the question, not of strength, but durability. This should be made as clear as possible. Durability in all these materials is based, therefore, on their composition, on their fibers, which may be analyzed as vegetable, animal, or mineral.

Burlap, for instance, is of a vegetable fiber (jute), the least durable of all the fibers, the most difficult, and one almost impossible to water-proof properly. It misleads architects and engineers into its use, because is seems so plausible to talk about its strength. This idea of strength, as the speaker has attempted already to explain, is a fallacy. Reinforcing concrete is based on an entirely different principle from attempting to reinforce asphalt with burlap. As already

Mr. De Knight.

explained, the essential thing in water-proofing is not strength, but to make it water-tight and durable. Burlap decays sooner than any of the fibers. Why use it, therefore, simply because of its temporary strength, *i. e.*, during application? Durability is thus being sacrificed for utility.

Cloth is also of a vegetable fiber (cotton), and a weak and also very perishable fiber. As a fiber *per se*, it can be easily water-proofed, but when twisted into a thread and woven into a cloth, the fabric shrinks, and it is difficult to get the water-proofing preservative, not only into the twisted threads, but also into the interstices. To accomplish this, the cloth is run through the saturating compound slowly. It is thus apt to be burned, and in this way there is destroyed at the beginning, not only its strength, but its durability and its real intrinsic worth.

The best material for water-proofing is a good felt composed of cotton and wool fibers—not the twisted, but the natural fiber. A good felt should contain from 25 to 30% of wool fiber. We know very well that wool is much more durable than cotton, and, therefore, that a mixture of wool and cotton produces a more lasting material than one which is composed entirely of cotton or vegetable fiber—such as cotton cloth.

Wool fiber or hair (animal fiber) is of a horn-like structure and contains oils which preserve it indefinitely.

With reference to asbestos paper. The speaker desires to be very frank in saying that he wishes to do all he can to warn against its use for water-proofing purposes. The following facts will be of interest and value to any one contemplating the use of this material for water-proofing, and should have earnest consideration.

To begin with, the fiber (so-called) in asbestos paper, is not a fiber. It is a splinter or sliver, a sloughing off lengthwise of the crude asbestos, which is the common mineral, hornblende. Hornblende contains silicate, magnesia, and lime or iron. The magnesia or lime in the asbestos tends to destroy any oil with which the paper is treated in water-proofing it. This explains why asbestos paper, saturated in asphaltic oil, dries out so soon and becomes dead.

The fiber or sliver itself is not a natural fiber; that is, it does not curl and felt into a natural felt like wool fiber with its many pellicles. Examined under a microscope, we find an asbestos fiber or sliver to be what is termed a straight-out or dead fiber. Therefore, to make it hold together in a paper, it must be artificially cemented together or sized with starch, tapioca, or something similar, making a paper absolutely lacking in flexibility and tensile strength. Dip the paper into water and it disintegrates by its own weight. The longer fiber or sliver of asbestos is so expensive that it is used in only the high-priced twine, rope, and asbestos cloth. It is too costly

Mr. De Knight.

to use in asbestos paper, which is composed of the weak and broken ends, or the refuse which cannot be used in twine and cloth. Therefore, the additional necessity of cementing the ingredients of this paper by sizing.

About fifteen years ago, the speaker was engaged in the manufacture of the first, so far as he knows, asphalt-treated asbestos paper made in the United States for use in below-grade water-proofing. It was first specified (in 1899-1900) by William Barclay Parsons, M. Am. Soc. C. E., then Chief Engineer of the New York Rapid Transit Subway Commission. The Subway was the first great work of its kind in the country. When Mr. Parsons set out to design and build it, there were no established guides. He had to establish new standards. There were no standard specifications for water-proofing—no treatises on the subject. "Modernized Waterproofing" by the speaker, in 1902, was the first of its kind. Asbestos paper treated with asphalt, was specified by Mr. Parsons for the original Rapid Transit Subway work, on the theory that as the mineral fiber or sliver from asbestos was fire-proof, it would be decay-proof, and, therefore, an ideal material for below-grade water-proofing. The theory did not work out in practice, however, because of the inherent defects in the fiber itself, as the speaker has already attempted to explain. This was not discovered until several thousand rolls of asbestos paper had been used in the work. The great trouble was that the material was so friable and weak that it tore easily when the workmen tried to handle it. In working it into corners or into angles formed between the floor and the wall, the fingers of the workmen easily went through the paper. Their shoes also went through it, when they walked on it. It died out rapidly when kept in storage. It could not be pulled, roughly handled, and manipulated like a wool-mixed coated felt. After investigation and the discovery of the real defect, which was inherent in the paper, its use was discontinued, and not a square foot of it was used thereafter.

From the standpoint of the material manufacturer, it matters little whether treated felt, treated cloth, treated burlap, or treated asbestos paper (if desired), is used, but from a technical and engineering standpoint, based on long and varied experience in the several branches —manufacturing as well as constructional—the speaker candidly recommends as the best material a saturated and coated felt containing from 25 to 30% of wool. It can be easily and thoroughly impregnated so as to preserve its every fiber. The felt should be coated, like enameled leather, on its two surfaces. This produces a tough, flexible, absolutely impervious sheet in itself, with no holes or interstices, as in cloth or burlap. The finished material is then as near like skin, hide, or leather—the most natural water-proofing—as it is possible to obtain. The standard method is to use from four to six layers

of such a coated, impervious, specially-made felt. It is cemented together with a bitumen compound as elastic as the felt. This forms in the mass a thick, tough, water-proof stratum. This stratum is apart from and practically independent of the concrete, which can settle, contract, expand, or crack without injuring this non-conducting stratum or membrane. It was because of the resemblance of such a specially prepared, glazed-coated felt to a skin or membrane that about ten years ago the speaker originated the expression "the membrane method". Mr. De Knight.

ALBERT H. RHETT,* ASSOC. M. AM. SOC. C. E. (by letter).—Some experiments made by the writer in connection with the construction of the bridges of the New York, Westchester and Boston Railway, may be of interest, because, after three winters' service, the results are entirely satisfactory. Mr. Rhett.

This work involved the water-proofing of 56 bridge structures, including 700 ft. of two-track viaduct over the Mount Vernon, N. Y., water supply, 2 500 ft. of four-track viaduct and the terminal station at 180th Street, New York City, two half-through railroad bridges where the under side of the bridge floor formed the ceiling of the station concourse, 1 mile of four-track subway, with two stations, and many crossings of important city streets and at station entrances. The problem, therefore, seemed to merit special attention, and as previous experience with standard methods did not furnish any great encouragement, it was determined, in sheer desperation, to make a somewhat radical experiment, though this experiment did not involve a departure from the elastic membrane theory, which was believed to be axiomatic in the water-proofing of live-load structures.

Observation had shown that there were two almost universal causes for the failure of a membrane in such structures, namely, the breaking away of the flashing of the membrane from the sides of the girders in half-through structures, due to the deflection of the floor, and the breaking of the membrane over the supports in multiple-span structures due to the deflection of the span.

It was thought that both these difficulties could be corrected if it were possible to devise a membrane, which, instead of being composed of layers of supposedly elastic water-proof fabric, should consist of a compound which would be permanently, and at all temperatures, adhesive, cohesive, elastic, and water-proof. If such a compound could be obtained, it would be self-healing when punctured; would stretch and not break over the supports; and, moreover, a permanently elastic water-proof connection could be made between this membrane and the side-supporting girders by a continuous trench in the concrete base, along the girders at their juncture, filled with this compound.

* Brooklyn, N. Y.

Mr. Rhett.

A careful search of the market did not reveal a compound which possessed these qualifications, and it became necessary to resort to the expedient of trying to have one manufactured. After several experiments this was accomplished and a substance was evolved, which, after a year's exposure to the open air, remained adherent, coherent, and elastic at all temperatures, adhering to glass at zero temperature and, though semi-plastic in its normal state, not liquefying except at high temperatures.

This substance, reinforced with burlap, was then used for a membrane, and the side trenches were partly filled with oakum on which the compound was poured. This composition has no destructive chemical effect on the burlap. As used, the membrane consisted of one heavy layer of the compound poured and swabbed on the surface of the floor, on which the burlap was laid and which, in turn, again received a light coat of the compound.

The method also presented a number of practical advantages. It was so easy of application that it could be done with unskilled labor; the compound being non-hardening, could be applied in cold as well as in warm weather; being permanently adhesive, it could be applied to a damp or wet surface; the side trench saved the usual elaborate protective fittings at the juncture of the membrane and the girder web; and, being permanently adhesive and semi-plastic, the cleaning of the surface of the girder, so carefully specified by Mr. Wagner, was entirely immaterial, and no attempt was made to clean this surface, except to brush off the loose dirt. The membrane was covered with a 3-in. layer of concrete reinforced with poultry netting.

This work was done in the winter and early spring of 1912, and in a letter to the writer, under date of January 25th, 1915, the Engineer of Maintenance of the Company, Mr. F. Zogbaum, states that "the water-proofing on all these bridges has proven very efficient".

All the structures, however, were not water-proofed by this method, quite a number of them having been treated with the standard 4-ply felt cemented together with the usual type of compound, and covered with hard-burned brick, run in with the same compound instead of cement mortar, and they have also proved satisfactory up to the present time.

In all cases an attempt was made to reinforce the floor proper, just beneath the membrane, against breaking, which would tend to tear the membrane, and this was done even in the trough floor structures. Longitudinal reinforcement in the form of expanded metal was inserted over all intermediate bents or columns; and over the abutments rods were used which were turned down over the back wall, the floor slab proper being always carried over the back wall, to a ledge thereon made to receive it.

Mr. Rhett.

With the exception of the 180th Street Viaduct, no attempt was made to drain the floors except over the back wall, and in few cases was the grade more than 1 per cent. In all cases, however, the water-proofing and its concrete cover, with the poultry netting, was carried over the back wall to the ledge on the back of the abutment. At the expansion end this was separated from the ledge by a layer of paper and compound, and a pad of corrugated asbestos about 2 in. thick was inserted between the face of the back wall and the turned-over portion of the floor slab, to permit of contraction, it being thought that the usual cause of leakage at this point was due to the tearing of the water-proofing by this contraction.

The Hutchinson River Viaduct, which is over the water supply of Mount Vernon, N. Y., is a two-track, solid-floor, deck structure, 700 ft. long, with two expansion joints, and as it was against the law to deliver the drainage on the adjoining property, it was necessary to let it flow the entire length of the structure on the roadbed alone, and though the grade was only 1%, and in spite of the two expansion joints, there has not yet been any complaint or trouble. This structure was water-proofed with the standard 4-ply felt and brick covering, and the expansion joint was made by a U-shaped, sheet-lead roll inserted in the 2-in. opening between the ends of the slabs of the sections, which was filled with oakum and the non-hardening, adhesive, cohesive compound. This opening was protected from the ballast by a steel cover-plate anchored to the slab of the fixed side of the opening.

The three special features of the system, as used, were:

First.—The use of a membrane of which the active water-proofing agent was an elastic compound instead of an elastic fabric.

Second.—The method of attaching the membrane to the face of the vertical girder by a pocket of a permanently adhesive and elastic compound, thus insuring a permanently elastic water-proof joint.

Third.—The special reinforcement, of the surface water-proofed, against cracking that might affect the integrity of the membrane.

As far as three years' service may constitute a test, the results have proven satisfactory.

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PAPERS AND DISCUSSIONS

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NOMOGRAPHIC SOLUTIONS FOR FORMULAS OF VARIOUS TYPES

Discussion.*

By Messrs. Allen Hazen and Carl B. Andrews.

Mr. Hazen.

Allen Hazen,† M. Am. Soc. C. E. (by letter).—This paper is important, as it calls attention to a method of easy and rapid graphical solution of many formulas, which is capable of wide application, especially where only three variables are involved. In the common hydraulic formulas there are four variables, namely, slope, radius, coefficient, and velocity (or quantity). Although the method can be extended to cover the four variables, much of its ease of application is lost when the fourth is added.

It is certainly true that much time is spent by some people in making computations for the production of diagrams which have little practical use. On the other hand, diagrams which are useful, can frequently be made. The diagram cited as a horrible example illustrates this. As presented, it was hard to make, and is of little use. With logarithmic paper, a parallel rule, and a few primary points, a more useful diagram could have been made with less effort.

The introduction of the log. log. slide-rule within the last few years has facilitated the solution of exponential formulas, and has permitted many calculations to be made rapidly and easily, which formerly could best be solved graphically on logarithmic paper.

The author shows the application of the nomographic method to the Williams-Hazen hydraulic-flow formula. The method is clearly

* This discussion (of the paper by R. C. Strachan, M. Am. Soc. C. E., published in December, 1914, *Proceedings*, and presented at the meeting of February 3d, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Hazen.

well adapted to this service, and is one of several that can be used.

Before this formula had been put in final form, the writer experimented with slide-rules of home-made construction. These were made by constructing slides a little smaller than those that came with an ordinary Manheim slide-rule, paper-covered, and graduated with the desired scale. In this way it was found that formulas of this type could be solved rapidly with sufficient accuracy. Afterward, a slide-rule was designed for the solution of this formula, a maker found, and several hundred such rules have been sold.

Engineers, like other people, have strongly fixed habits. Some engineers can best solve their problems on a slide-rule, some by the use of diagrams, and others by printed tables. For the solution of the Williams-Hazen hydraulic formula, at least five procedures may be used: (1) the hydraulic slide-rule, made by Ledder and Probst, of Boston; (2) "Hydraulic Tables", published by John Wiley and Sons; (3) diagrams on logarithmic paper, of which there are a number of very excellent ones, such as those published in some of the engineering papers; (4) the procedure suggested by the author; and (5) direct solution with a log. log. slide-rule.

The writer believes that those who have used the slide-rule sufficiently to become accustomed to it would not be willing to adopt any of the other methods. Getting accustomed to the slide-rule is not difficult, but many engineers are reluctant even to make this effort, and it is interesting to note that eight times as many copies of the "Hydraulic Tables" have been sold as of the slide-rules. Also, notwithstanding the existence of both the slide-rule and the tables, several members of the Society have thought it worth their while to make special diagrams on logarithmic paper for the solution of the formula. The nomographic method will no doubt appeal to others, and will be useful to them.

By calling attention to an excellent method of solving formulas of this type rapidly, the author has rendered a substantial service to the Profession.

Mr. Andrews.

CARL B. ANDREWS,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Strachan remarks in his synopsis that the principle of the isopleth has not received from American engineers that degree of attention to which it is entitled. This statement is undoubtedly true, for the alignment diagram has all the advantages over diagrams platted by rectangular co-ordinates, which Mr. Strachan claims, and more.

Some years ago the writer read part of M. d'Ocagne's treatise on Nomography, and became interested to the extent of drawing a number of alignment diagrams, and writing a paper on the subject, which was read before a local engineering society. Of the diagrams produced, however, only two have ever been used enough to justify the

* Honolulu, Hawaii.

time spent in making them, these two being intended for the reduction of stadia readings to vertical and horizontal distances. It seems to the writer that many of the mathematical operations which occur in civil engineering work require a higher degree of accuracy than is obtainable from diagrams of any kind, and that the diagrams find their proper use in checking computations and in making quick approximate calculations. Mr. Andrews.

A class-room remark of the late Professor Thomas Gray, which strongly impressed the writer, was to the effect that there was a disadvantage in the use of reckoning machines, diagrams, multiplication tables, and other such mechanical aids in computation, in that the computer was likely to get into the habit of writing down the results without noting the intermediate steps, which made it very hard to trace errors resulting from an accidental wrong entry or wrong reading. In the preparation of the Smithsonian Physical Tables, Dr. Gray said that much computation had been required, but that comparatively few errors had been found in the work after its publication, because the computations had been carried out in full in note-books, where they were readily checked.

In offices where there is a great deal of computation, of a kind in which diagrams may be used to advantage, alignment diagrams, as described by Mr. Strachan, are undoubtedly much more convenient and useful than those platted by rectangular co-ordinates.

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DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS*

By Dugald C. Jackson, M. Am. Soc. C. E.

Mr. Jackson.

Dugald C. Jackson,† M. Am. Soc. C. E. (by letter).—This report contains much matter for reflection on the part of engineers who desire to see the condition of the profession improved financially, and also in established reputation before the public. The report must be recognized as being incomplete as yet, but the statistics which are being gathered are leading to light in respect to at least the financial conditions of engineers, and may be expected ultimately to give valuable information on various conditions relating to their employment.

The report has some encouraging features, as it shows that among men who have been long enough in the profession to become soundly established (as, for instance, 20 years) at least half are obtaining incomes of income tax standard; but it is discouraging to observe that at least 50% of the engineers of several decades of experience and presumably steady, conscientious work, never attain an income from their professional employment that exceeds $4 000 or $5 000. In other words, the line of yearly compensation of the middle-man on the chart, Plate XL, shows that 50% of all the engineers of long years of training and experience are never able to go above such compensation. The writer suggests that one of the most important points for this Committee to investigate, if means can be devised for such an investigation, is the reasons for this low status, financially, of so many men in the profession. A comparison should also be made with other professions, if practicable. It may be assumed that members of the Ameri-

* This is a discussion on the Progress Report of the Special Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers, presented to the Annual Meeting, January 20th, 1915.

† Boston, Mass.

Mr. Jackson.

can Society of Civil Engineers are picked men, out of the Engineering Profession at large, and therefore that the average financial status of the members of the profession is even less desirable than is shown in these charts.

The diagram, Fig 1, perhaps indicates the effect of substantially a life tenure on those who are in the employ of governmental corporations, technical schools, and railroads, as compared with employment in private companies and with consulting engineers or contractors, with whom life positions and pension arrangements are less common. A question here arises that the Committee might consider, namely: Is there, in fact, justice in making so much difference as is indicated in this diagram? Obviously, a substantial life tenure under the equivalent of a civil service rule, with a pension arrangement at the end, justifies some reduction of yearly compensation, as compared with employment under more strictly competitive conditions; but, again, does such life tenure interfere with the development of the highest ideals of initiative and resourcefulness in engineering?

The third diagram, Fig. 2, shows a marked difference between the status of the older engineers in New England and in other parts of the country. Inasmuch as the number of engineers reporting for the 31 to 35 and the 36 to 40-year periods in New England are comparable with the numbers reporting for the other divisions of the land, except the Middle Atlantic territory, it would appear that the better financial status of the older men in the profession in New England may have some significance, and this again is a point that is worthy of close scrutiny by the Committee, as it may bring light which may serve to aid the status of engineers in other parts of the country.

The table on page 3068* cannot be accepted as giving adequate information. It seems to be on a different basis for engineering school graduates than that for non-graduates. The laws of the Society admit the engineering school course as the equivalent of a certain period of practice (two years), and, under these circumstances, the column giving the average age of graduates in proportion to years of experience apparently ought to be dropped two lines to make it correspond with the column respecting the age of non-graduates. If this is done, it changes the conclusions in regard to age stated immediately below the table. If the modification thus suggested is reasonable and appropriate, it also affects the relationships of the curves on Fig 3, and it would perhaps indicate that there is not as early a preference in the compensation of graduates as the chart shows, but there is a continued betterment of compensation for the graduates in after years.

In respect to Fig. 3, the table printed thereon is obviously one which should not appear in association with a chart of this kind, or else the chart itself should be explained more fully. This chart seems to be

* *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Jackson.

objectionable on account of being incomplete, since it gives its information in such a fashion that its meaning cannot be derived from its face. This diagram has the same fault as the main chart of the Progress Report of 1914, which did not include the line indicating the status of the median man. The statistics on this diagram are presented in such form that they are not clearly indicative of the facts, and are therefore likely to give an erroneous impression. Such inadequate presentation of statistics seems to be out of harmony with the best ideals of engineering, and the writer believes it would be preferable to eliminate this chart from the report until the Committee has had an opportunity to study the detailed figures more closely and devise means for representing them in a more adequate fashion.

The work of this Committee seems to be well worth extending, as the report suggests. The Society has given much attention to questions of engineering ethics, to questions of the fidelity of engineers to their employments, and to questions of engineering standards. It can properly give more attention than heretofore to the conditions under which engineers are employed. If the Committee can be continued under the efficient guidance of its Chairman, Mr. Lewis, it may be hoped that additional light may be had on this important subject.

It should be remembered, especially, that this is a Progress Report by a committee which has to deal, not only with the compensation of engineers, but also, generally, with the conditions of their employment. The matter of compensation is an aspect of the general question in which statistics can be most easily gathered, and it is therefore the natural starting point for the work of such a committee.

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DISCUSSION ON STEEL COLUMNS AND STRUTS*

By William W. Crehore, M. Am. Soc. C. E.

Mr. Crehore.

William W. Crehore,† M. Am. Soc. C. E.—The thanks of the Society are due to this Committee for the very efficient and thorough way in which it has designed these tests and is carrying them out. They are certainly the most important column tests that the Profession has ever witnessed.

As bearing on what is being done and what yet remains to be done, it is perhaps proper at this time to call attention to the fact that the Committee's programme seems to provide for the testing of small columns only. The sections listed in this Progress Report show values of the radius of gyration varying from 2.24 to 2.51, which indicate very much smaller columns than the average section in common practice. The average section in practice has a radius of gyration between $3\frac{1}{2}$ and $6\frac{1}{2}$, so that it seems evident that these tests as at present prepared will not be fairly representative. Is it to be argued from the behavior of these small columns under test that columns of larger section having the same form and subject to similar conditions of stress will behave the same whenever the value of $\frac{l}{r}$ is the same? It seems to the speaker that this is one of the things the Profession is trying to find out. For many years we have had our column formulas and diagrams based on isolated tests made under various conditions. There is need for definite information on the point whether or not a formula based on tests of small sections with small radii of

* This is a discussion on the Progress Report of the Special Committee on Steel Columns and Struts, presented to the Annual Meeting, January 20th, 1915.

† New York City.

Mr. Crehore.

gyration will be a proper one to use for the larger sections in average practice.

The limits of the testing machine, 1 150 tons in compression and 33 ft. in length, will permit the test to destruction of a column with radius as large as 3.3 in., having any one of the three values of slenderness adopted by the Committee. But such a column would not be representative of common practice, either. Tests to destruction might be made, however, at 85 and 50 radii, of columns having a radius as large as 4.65, and at 50 radii of columns having a radius as large as 6.37. Comparisons of the tests on such sizes, even without the high values of slenderness, would be very valuable, taken in connection with the complete range of tests of the small sizes, as indicating whether or not the large column does behave the same as the small one under similar conditions.

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PROGRESS REPORT OF THE SPECIAL COMMITTEE TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS*

To the American Society of Civil Engineers:

Your Special Committee, appointed "To Codify Present Practice on the Bearing Value of Soils for Foundations, and Report upon the Physical Characteristics of Soils in Their Relation to Engineering Structures", was authorized by the Board of Direction, January 7th, 1913, and now submits this, its report of results achieved, a programme for the coming year, and a forecast of work in contemplation.

The last report of your Committee was a review of its subject matter, which is divided into two parts:

1.—Present practice on the bearing value of soils;

2.—Physical characteristics of soils in relation to engineering structures.

Although the membership of your Committee is widely distributed, a number of meetings have been held during the past year.

Your Committee, recognizing the many inconsistencies in present practice, is collecting information from every available source. In consequence its Circular No. 1, asking for information and data relating to tests, data, and local practice, is about to be forwarded to all members of the Society. Replies, as received, will be tabulated in a manner convenient for record, study, and subsequent analysis. Samples of the soils reported on in the replies to this circular are solicited for the purpose of identifying and comparing them with the Committee's proposed classification. A number of samples have already been received.

Your Committee reports that, although scattered information exists, no previous effort seems to have been made by engineers to

* Presented to the Annual Meeting, January 20th, 1915.

classify soils or study their characteristics. Hence, the progress of your Committee has been difficult and deliberate. The distrust heretofore shown toward a scientific investigation is perhaps due to a failure to recognize the fundamental differences in soils. Isolated bearing tests, which have been made on soils assumed to be the same, have produced results which cannot be co-ordinated.

Your Committee has thoughtfully considered the classification of the soils of the United States for the study of their physical properties. Soil may be said to be the unconsolidated veneer covering the rock crust of the earth. It is derived from decomposed and disintegrated rocks. The physical constituents of the soil are small pieces of rock, stones, sand, silt, and clay, all of which, except clay, have no individual coherence. Clay in one form or another may be said to be the matrix that binds the particles of soil together and renders it plastic, and cohesive.

As each soil consists of particles of various sizes and grades, the relative proportion of the different sizes of the particles determines its "texture", and the structural arrangement of the particles constitutes its "structure".

Differences in the physical qualities of soils may be traced to the rock from which they have been derived, but mainly to their process of formation. Those derived from the degeneration of the underlying rocks are residual soils, whereas those formed from material transported since it was broken down are colluvial, alluvial, æolian, or glacial, according to the agency of transportation. The general processes of formation of soils are moisture and temperature. Rainwater decomposes the constituents of rocks by inducing chemical changes, and variations of temperature disintegrate the mineral aggregates of the rock, through expansion by heat and contraction by cold. The complex nature of soils appears to be due to the variety of influences involved in their formation, rather than in the material from which they are derived.

A satisfactory classification must be based on the inherent and invariable properties of the material. Though climate, geology, etc., have been carefully considered by others for a basis of classification, from an engineering standpoint, they are unsatisfactory. Texture being determinate is one property that can be used as a basis of classification. Consequently, soils may be divided arbitrarily into classes, according to their texture, each forming an individual class depending on the sizes or separates that predominate. As there may be a number of types of the same texture, other differences must be recognized, such as the property of "structure", by which soils may be considered as porous or impervious, loose or compact, in their structural arrangement. However, your Committee has in use a tentative

classification based on the textural factor, and is giving the subject further study.

In connection with the textural classification, your Committee found it necessary to investigate various methods for the mechanical analysis of granular material. The results as to methods and apparatus previously in use were found to be so conflicting that an entirely revised, practical, and consistent procedure has been instituted for trial. In the meantime, suggestions for the separation of fine particles will be welcomed by your Committee.

In order to secure comparable results in sifting the sizes of the larger particles, the adoption of screens with uniform opening became necessary. After a very thorough investigation your Committee is using the uniform screen scale and specifications described in Appendix A. These screen openings are based on a geometrical ratio of $\sqrt{2}$, increasing uniformly from the United States Standard No. 200 screen to the No. 8°. The manner of recording, tabulating, and platting mechanical analyses is subject to further consideration.

The complete study of the physics of soils requires that samples be secured as nearly as possible in an undisturbed state. For this purpose a sampler has been designed and used successfully in sandy clays, but requires some modifications for other types of soils. When tested and proven, the drawings will form part of the final report of your Committee.

Last year your Committee solicited the co-operation of the U. S. Government Departments at Washington, and it has pleasure in announcing that Dr. S. W. Stratton, Director of the U. S. Bureau of Standards, has appointed a committee of scientists, with facilities and funds, and that this committee is now actively engaged in an investigation for the determination of the properties of earths in relation to the sustaining power of piles, foundations, effects of moisture, viscosity effects on time rate of depression under loads, standardization of soils for the structural purposes of engineering, etc. For the present, experimentation will be made to determine the laws of variation of different coefficients throughout plastic and pulverulent media under suitable strain, especially the coefficients of friction, the laws of variation of lateral pressure with the vertical, the variation of the shearing forces, volumetric compression, effects of moisture content, etc., as preliminary to the study of the more formal problems of engineering, such as piles, foundations, and retaining walls. Your Committee also begs to state that Dr. J. A. Holmes, Director of the U. S. Bureau of Mines, has appointed a committee which is studying pressures in tunnels and other subsoil operations; and Dr. Logan Waller Page, Director of the U. S. Department of Public Roads, has offered the services and facilities of his office, which have been accepted. Your Committee is in consultation with these Govern-

ment institutions, and here expresses its expectations that their assistance will be of much help in the solution of its problem. Your Committee also acknowledges with much appreciation the assistance of the Technological Branch of the Carnegie Library of Pittsburgh, through the courtesy of Mr. Harrison Craver, Librarian, and Mr. J. C. McClelland, Technology Librarian, in the preparation of an up-to-date bibliography on its problem having nearly four hundred references, which are given as Appendix B.

During the coming year your Committee expects to continue the study, and will endeavor to complete the basic work, of the classification of soils and the methods of sampling and mechanical analysis, and codify the information on tests and present practice obtained from the circulars. Through sub-committees of its members, studies have been commenced on earth pressures, land slides and erosion, roadbed, and bearing piles.

In view of the extent of the programme, no definite forecast can be fixed for the time of completion of the final report of your Committee, and it requests to be continued.

Respectfully submitted for the Committee, January 20th, 1915.

ROBERT A. CUMMINGS,
Chairman.

COMMITTEE:

W. J. DOUGLAS,
EDWIN DURYEA, JR.,
J. C. MEEM,
FRANK M. KERR,
E. C. SHANKLAND,
SAMUEL T. WAGNER,
ROBERT A. CUMMINGS.

APPENDIX A

UNIFORM SCREEN SPECIFICATIONS

The uniform screens shall be of wire cloth, plain woven (not twilled) from brass, bronze, or other suitable wire, and mounted on frames, without distortion. Obvious imperfections in the screen cloth or mounting, as, for example, punctured, loose, or wavy cloth, imperfections in soldering, etc., will be cause for rejection. Wire coarser than 0.070 in. in diameter shall be made with an extra crimp between the wires. The diameters of the wires shall be as given in Column III of Table 1, and the average diameter of such wires as may be measured shall not be outside the limits given in Columns IV and V.

The number of warp or short wires per whole inch, as measured at any point of the screen, shall not be outside the limits given in Columns VI and VII, and for screens coarser than No. 20 those given in Columns VIII and IX. Screen No. 3 and finer screens can be identified by the number of openings per inch equal to the screen number.

TABLE 1.—Uniform Screen Specifications.

Screen Scale Ratio: 1.414 = 2.

Screen No.	Openings, in inches.	Diameter of wire, in inches.	Limits of Wire Diameters.		Limits for Number of Openings for any Whole Inch Interval.		Limits for any Openings, in Inches.	
			Warp or shoot, in inches.		Warp.			
I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.
8°	4.198	0.375	0.376	0.374	.....	.....	4.218	4.178
7°	2.969	0.207	0.208	0.206	.....	.....	2.989	2.949
6°	2.099	0.192	0.193	0.191	.....	.....	2.114	2.084
5°	1.485	0.149	0.1498	0.1482	.....	.....	1.495	1.475
4°	1.050	0.149	0.1498	0.1482	.....	.....	1.060	1.040
3°	0.742	0.135	0.1358	0.1342	.....	.....	0.752	0.732
2°	0.525	0.105	0.1058	0.1042	.....	.....	0.535	0.515
1°	0.371	0.092	0.0928	0.0912	.....	.....	0.381	0.361
3	0.263	0.070	0.0707	0.0693	.....	.....	0.270	0.256
4	0.185	0.065	0.0657	0.0643	4.05	3.95		
6	0.131	0.036	0.0366	0.0354	6.05	5.95		
8	0.093	0.032	0.0326	0.0314	8.125	7.875		
10	0.065	0.035	0.0355	0.0345	10.125	9.875		
14	0.046	0.025	0.0255	0.0245	14.25	13.75		
20	0.0328	0.0172	0.0177	0.0167	20.5	19.5		
28	0.0232	0.0125	0.0129	0.0121	28.5	27.5		
35	0.0164	0.0122	0.0126	0.0118	35.5	34.5		
48	0.0116	0.0092	0.0095	0.0089	49.0	47.0		
65	0.0082	0.0072	0.0075	0.0069	66.0	64.0		
100	0.0058	0.0042	0.0045	0.0039	104.0	96.0		
150	0.0041	0.0026	0.0028	0.0024	156.0	144.0		
200	0.0029	0.0021	0.0023	0.0019	208.0	192.0		

Measurements of Screens.—The average diameter of the wires shall be obtained by measuring a sufficient number with a micrometer microscope. Measurements made with the micrometer caliper will be sufficiently accurate for wires or openings having a diameter of 0.025 in. or larger. No. 20 screen and finer shall be measured with a graduated glass scale laid on the screen, readings being taken over several inch intervals. In lieu of the foregoing requirements, the No. 200 screen will be accepted when approved by the U. S. Bureau of Standards as having a screening value equal to that maintained by the Bureau of Standards.

APPENDIX B

BIBLIOGRAPHY OF PHYSICAL PROPERTIES AND BEARING VALUE OF SOILS

This bibliography is by no means complete. Many more references are to be added, and in its final form a logical classification will be attempted. The present arrangement is alphabetically by authors. Titles of articles and names of journals are given in full, and dates, volume numbers, and inclusive paging are indicated. In most cases the nature of the article is indicated by a brief explanatory note.

ABBOTT, HUNLEY. Discussion of the Carrying Capacity of Bulb-pointed Concrete Piles. 1911. (*Engineering and Contracting*, v. 35, pp. 41-43.) Takes into consideration different kinds of soils and "safe bearing power of soils."

ABBOTT, HUNLEY. Method of Constructing Gas-holder Foundations in Soft Soils, with Some Costs. 1912. (*Engineering and Contracting*, v. 37, pp. 199-200.)

ACTUAL RESISTANCE OF BEARING PILES. 1893. (*Engineering News*, v. 29, pp. 171-173.) Records, with notes, 17 different examples of actual weight supported by piles at different places.

ADAMS, FRANK D., and COKER, ERNEST G. Investigation into the Elastic Constants of Rocks, more Especially with Reference to Cubic Compressibility. 69 pp. 1906. (Carnegie Institution of Washington. *Publication No. 46.*)

——6 diag., 6 dr. 1906. (*American Journal of Science*, v. 172, pp. 95-123.)

——Abstract. 1907. (*Beiblätter zu den Annalen der Physik*, v. 31, pt. 1, pp. 186-187.)

——Abstract. 1907. (*Minutes of Proceedings*, Inst. C. E., v. 169, pp. 476-477.) Measurements were taken of the longitudinal contraction and lateral expansion under a longitudinal stress, from which data all the elastic properties were estimated.

AIRY, WILFRID. On the Slopes of Cuttings. 1879. (*Minutes of Proceedings*, Inst. C. E., v. 55, pp. 241-251.) Mathematical discussion on friction of soils.

ALLAIRE, ALEXANDER. Foundation Work in Montreal. 1 dr., 4 ill. 1912. (*The Canadian Engineer*, v. 22, pp. 188-190.) Considers, in general, quality of soils underlying Montreal, and types of foundations constructed for different structures.

ALLEN, KENNETH. Design of Retaining Walls. 1892. (*Engineering Record*, v. 26, pp. 341-342, 356-357, 374, 393.) A practical design of retaining walls, sea walls, and dock walls. Illustrated with actual designs.

ALLEYNE, SIR JOHN GAY NEWTON. Dordrecht Railway Bridge; and the Foundations of the Railway Bridge at Rotterdam. 1875. (*Minutes of Proceedings*, Inst. C. E., v. 42, pp. 213-227.) Gives formula for load on piles used in Holland, pp. 216-220.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION. Report (of Committee) on Retaining Walls and Abutments. 1909. (*Proceedings*, Tenth Annual Convention, Am. Ry. Eng. and Maintenance of Way Assoc., pp. 1317-1337.)

——Condensed. 1909. (*Engineering Record*, v. 60, pp. 288-290.) Gives information showing practice of various railroads in the designing of retaining walls. Committee submits method of determining earth pressures based on Rankine's formulas.

AMERICAN RAILWAY ENGINEERING ASSOCIATION. Report (of Committee) on Unit Pressures Allowable on Roadbeds of Different Materials. 1912. (*Proceedings*, Thirteenth Annual Convention, Am. Ry. Eng. Assoc., pp. 388-396.) Discusses bearing powers of soils of different kinds, as found by previous investigators.

ANDERSON, WILLIAM. Antwerp Water-Works. 1883. (*Minutes of Proceedings*, Inst. C. E., v. 72, pp. 24-44.) Discussion, pp. 44-78. Foundations, pp. 37-38, 53-55. Different ways of building foundations are discussed, also supporting power of pile as calculated by Sanders' formula.

ANDREWS, HORACE. Earth Settlement in City Streets. 1906. (*Municipal Engineering*, v. 31, pp. 361-366.) Paper before American Society of Municipal Improvements. Largely a compilation of material that has previously appeared, on the properties and behavior of clayey soils.

ANGLIN, S. Foundations of Buildings. 1899. (*Journal*, Royal Inst. of British Architects, v. 49, pp. 14-18.)

——Abstract. 1899. (*Engineering Record*, v. 40, pp. 679-680.)

——1900. (*Stone*, v. 21, pp. 243-250.) Paper before Manchester Society of Architects. Considers what foundation soils may be looked on as reliable substrata. Discusses desirable working pressures for different clays, gravels, and sands.

APJOHN, JAMES HENRY. Note on the Movement of the Walls of the Kidderpur Docks. 4 dr. 1895. (*Minutes of Proceedings*, Inst. C. E., v. 121, pp. 104-151.) With discussion. The precarious nature of clay soil is brought out in discussion by W. R. Galbraith, Sir Douglas Fox, and F. E. Robertson, pp. 114, 125, 149.

AUS, GUNVALD. Reinforced Wall Foundations on Yielding Subsoil. 800 w. 3 dr. 1908. (*Engineering News*, v. 60, p. 5.) Briefly touches on yielding properties of light soils, and favors use of continuous foundation under walls heavily reinforced with longitudinal and transverse steel or reinforced concrete beams.

BAILLAIRGE, C. Supporting Power of Piles. 1902. (*Engineering Record*, v. 45, pp. 183-184.) Letter to editor discussing defects of existing formulas and suggesting a board of inquiry into the subject of pile foundations by American and Canadian Governments.

BAILLY, THOMAS C. J. Determination of Actual Earth Pressure from a Cofferdam Failure. 1906. (*Engineering News*, v. 56, p. 170.)

BAINBRIDGE, F. H. Methods and Costs of Testing for Bridge Foundations, 1908. (*Engineering-Contracting*, v. 30, pp. 352-354.) Describes making of borings to determine character of foundation. Refers particularly to work on Chicago and Northwestern Railway Bridge.

BAKER, BENJAMIN. Actual Lateral Pressure of Earthwork. 41 dr. 1881. (*Minutes of Proceedings*, Inst. C. E., v. 65, pp. 140-184.) Index of examples and figures, pp. 185-186. Discussion, 4 dr., pp. 187-208. Correspondence, 4 dr., pp. 209-241.

——1881. (*Van Nostrand's Engineering Magazine*, v. 25, pp. 333-342, 353-371, 492-505.) Author analyzes a few dozen experiments that were performed by him, and comes to the conclusion that the laws governing the lateral pressure of earthwork are not satisfactorily formulated.

BAKER, IRA O. Treatise on Masonry Construction. ed. 8. 1898. 556 pp. Section on "bearing power of soils", pp. 188-199. Considers different kinds of soils, and gives figures and calculations, and means for improving the bearing power. "Bearing power of piles," pp. 233-250.

BARBOUR, FRANK A. Strength of Sewer Pipe and the Actual Earth Pressure in Trenches. 2 diag., 3 dr. 1897. (*Journal*, Assoc. Eng. Soc., v. 19, pp. 193-241.)

——Abstract. 1898. (*Minutes of Proceedings*, Inst. C. E., v. 132, p. 409.) Gives results of experiments, and derives analytical relation.

BARLOW, WILLIAM HENRY. Description of the Clifton Suspension Bridge. 1867. (*Minutes of Proceedings*, Inst. C. E., v. 26, pp. 243-257.) Gives limiting load on brickwork, p. 248.

BARTLETT, JAMES. Foundations. 1910. (Encyclopædia Britannica, ed. 11, v. 10, pp. 738-743.) Discusses safe loads for soils of different characters, and making of trial borings.

BAUMANN, FREDERICK. Foundations. 1898. (*Inland Architect*, v. 32, pp. 42-45.) Gives figures and estimates for safe loads for rock, gravel, gravel and clay, sand, and clay. Considers nature of Chicago soil, and adequate loads.

BAUMGARTEN, KARL. Thunder Mountain Landslide. 1910. (*Mining and Scientific Press*, v. 101, pp. 698-699.) Describes progress of landslide or mud flow.

BEARING POWER OF PILES. 1894. (*Engineering News*, v. 31, pp. 283-284.) Tests were made while driving piles for foundations of Chicago Public Library. Bearing power of soils is considered, and formulas tested for bearing power of piles.

BEARING TESTS FOR HEAVY FOUNDATION LOADS. 1 dr. 1909. (*Engineering Record*, v. 60, p. 55.) Describes method, and gives results of test of the supporting power of soil composed of fine compact sand together with small pebbles and gravel.

BENABENG, J. Resistance des Pieux: Theorie et Applications. 16 diag., 29 dr. 1911. (*Annales des Ponts et Chaussées*, ser. 9, v. 5, pp. 263-355; ser. 9, v. 6, pp. 475-543.)

——Abstract. 1912. (*Minutes of Proceedings*, Inst. C. E., v. 158, pp. 475-476.) Deals with the bearing power of piles from a mathematical standpoint. The first part treats the subject from a new standpoint, namely, that of statical equilibrium. The second part follows more or less normal lines. Numerous formulas are deduced. Gives many tables of constants.

——Abstract. 1912. (*Engineering Record*, v. 65, p. 248.)

——Condensed. 1912. (*Le Génie Civil*, v. 60, pp. 246-250.)

BIHLER, C. S. Strength of Piles. 1 diag. 1904. (*Railway and Engineering Review*, v. 44, pp. 206-209.)

——Abstract. 1904. (*Minutes of Proceedings*, Inst. C. E., v. 158, pp. 475-476.) Gives results of test of long timber piles. (Note: In this publication the author's name is given as Bilder.)

BLACK, R. P. Remedies for Landslides and Slips on the Kanawha and Michigan Railway. 3 dr., 1 plate. 1911. (*Transactions*, Am. Soc. C. E., v. 71, pp. 1-10.) With discussion.

BLYTH, EDWARD LAWRENCE IRELAND. Description of the Loch Ken Viaduct, Portpatrick Railway. 1862. (*Minutes of Proceedings*, Inst. C. E., v. 21, pp. 258-264.) Gives data on load supported on piers, p. 260.

BOARDMAN, H. P. Concerning Retaining Walls and Earth Pressures. 1905. (*Engineering News*, v. 54, pp. 166-169.) Concludes that information regarding earth pressures is quite inexact. Suggests carrying out series of tests on large scale.

BOERNER, FRANZ. Künstliche Fundierung des Geschäftsgebzau des für das Oberlandesgericht zu Düsseldorf. 1908. (*Beton und Eisen*, v. 7, pp. 340-343, 360-364.) First part considers underlying soil strata, which were made up at successive depths of (1) filling, (2) layer of alluvial earth consisting of loose sand and fine blue sand mixed with a blue clayey material, and (3) a layer of coarse-grained gravel. Loading tests are given for the different soils, with use of piles.

BONE, EVAN P. Reinforced Concrete Retaining Wall Design. 1907. (*Engineering News*, v. 57, pp. 448-452.) Calculations of earth pressures, and diagrams.

BOUSSINESQ, J. Note sur la Determination de l'Epaisseur Minimum que doit avoir un Mur Vertical, d'une Hauteur et d'une Densité Données, pour Contenir un Massif Terreux, sans Cohesion dont la Surface Supérieure est Horizontale. 1 diag. 1882. (*Annales des Ponts et Chaussées*, ser. 6, v. 3, pp. 625-643.) Shows the application of the theory of earth-pressures as developed by Rankine and Darwin to design of vertical walls.

BOUSSINESQ, J. Note on Mr. G. H. Darwin's Paper "On the Horizontal Thrust of a Mass of Sand. 1882. (*Minutes of Proceedings*, Inst. C. E., v. 72, pp. 262-271.) Attempts to show that Mr. Darwin's conclusions are very similar to his own as expounded in his "Essai Theorique sur l'Equilibre des Massifs Pulverulents," Paris, Gauthier-Villars. 1876. Mr. Darwin's paper is in *Minutes of Proceedings*, Inst. C. E., v. 71, p. 350.

BRAWNING, C. Gravel as Ballast. 1912. (*Proceedings*, Thirteenth Annual Convention, Am. Ry. Eng. Assoc., v. 13, pp. 267-289.) Translated from *Zeitschrift für Bauwesen*, 1904. Considers bearing capacity of gravel particles.

BRICK AND IRON FOUNDATION FOR A GERMAN BUILDING. 800 w. 1898. (*Engineering Record*, v. 38, p. 9.) Alluvial clay stratum with supporting power of about 3 200 lb. per sq. ft., makes up foundation soil.

BRIGGS, LYMAN J. Electrical Instruments for Determining the Moisture, Temperature, and Soluble Salt Content of Soils. 35 pp. 1899. (*Bulletin No. 15*, U. S. Soils Division.) Describes in detail the electrical soil hygrometer, thermometer, and electrolytic bridge. Gives methods of standardization and practical operation.

BRIGGS, LYMAN J. Mechanics of Soil Moisture. 24. pp. 1897. (*Bulletin No. 10*, U. S. Soils Division.) Among other topics, discusses the capacity of soil for water, adjustment of water between a dry and a wet soil, and the relation of texture, structure, and temperature to water capacity.

BRIGGS, LYMAN J., and McLANE, JOHN W. Moisture Equivalents of Soils. 23 pp. 1907. (*Bulletin No. 45*, U. S. Dept. of Agriculture, Soils Bureau.) Deals with a method of determining the quantity of water which different soils are capable of retaining when the soil moisture is subjected to a constant measured force sufficient in magnitude to remove the moisture from the larger capillary spaces.

BRIGGS, LYMAN J., and others. Centrifugal Method of Mechanical Soil Analysis. 38 pp. 1904. (*Bulletin No 24*, U. S. Dept. of Agriculture, Soils Bureau.) Also briefly discusses other methods of mechanical soil analysis.

BRUNLEES, JAMES. Description of the Iron Viaducts Erected Across the Tidal Estuaries of the Rivers Leven and Kent, in Morecambe Bay, for the Ulverstone and Lancaster Railway. 1 pl. 1858. (*Minutes of Proceedings,* Inst. C. E., v. 17, pp. 442-447.) Treats of bearing power of quicksands.

BUCKINGHAM, EDGAR. Studies of the Movement of Soil Moisture. 61 pp. 1907. (*Bulletin No. 38,* U. S. Department of Agriculture, Soils Bureau.) Mostly devoted to capillary action in soils.

BUILDING AND MACHINERY FOUNDATIONS IN QUICKSAND. 2 dr. 1906. (*Engineering Record,* v. 53, pp. 247-248.) Fourteen-story office building, with pile foundation.

BURR, WILLIAM A. Resistance of Soils to Erosion by Water. 1894. (*Engineering News,* v. 31, p. 124.) Shows that value of soils for hydraulic purposes depends on proportions of sand and clay, the clay being better able to resist erosion than sand.

BURSTING PRESSURE OF AN EARTH FILL. 1912. (*Engineering News,* v. 68, pp. 593-594.) Editorial discussing the causes of failure of a retaining wall in St. Louis.

CAIN, WILLIAM. Earth Pressure. 1882. (*Van Nostrand's Engineering Magazine,* v. 26, pp. 89-104.) Tests formula to find how closely practice confirms theory.

CAIN, WILLIAM. Experiments on Retaining Walls and Pressures on Tunnels. 21 diag. 1911. (*Transactions,* Am. Soc. C. E., v. 72, pp. 403-448.) Discussion, 6 diag., pp. 449-474.

——Abstract. 1911. (*Minutes of Proceedings,* Inst. C. E., v. 185, p. 398.) Discusses a large number of experiments and derives several conclusions.

CAIN, WILLIAM. Cohesion and the Plane of Rupture in Retaining Wall Theory. 1 diag. 1912. (*Engineering News,* v. 67, p. 992.) Letter to editor discussing M. Hirschthal's article "Some Contradictory Retaining Wall Results," *Engineering News,* v. 67, p. 799.

CAIN, WILLIAM. Retaining Walls. 1880. (*Van Nostrand's Engineering Magazine,* v. 22, pp. 265-277.) Theoretically considers "the earth as a homogeneous and incompressible mass, made up of little grains, possessing the resistance to sliding over each other called friction, but without cohesion."

CALCULATIONS FOR RETAINING WALLS. 1911. (*Architect and Contract Reporter,* v. 86, pp. 43-44, 59-61, 75-76, 85-87, 96-97, 109-110.) Takes all factors into consideration, wind pressure, slides, earth pressure, etc. "Angles of repose of various earths," p. 109.

CAMBRIA STEEL COMPANY. Safe Bearing Capacity of Soils, etc., in Tons per Square Foot. 1914. ("Cambria Steel," a handbook of information relating to structural steel, ed. 11, p. 310.) Table giving bearing capacity of different soils for 29 different cities in the United States.

CAMERON, FRANK K. Dynamic Viewpoint of Soils. 1909. (*Journal of Industrial and Engineering Chemistry,* v. 1, pp. 806-810.) A criticism of the static theory of soils.

CAMERON, FRANK K., and BELL, JAMES M. Mineral Constituents of the Soil Solution. 70 pp. 1905. (*Bulletin No. 30,* U. S. Dept. of Agriculture, Soils Bureau.) Treats of soil in general, of minerals in the soil and their solubility, giving the effects of various agencies on solubility.

CAMERON, FRANK K., and GALLAGHER, FRANCIS E. Moisture Content and Physical Condition of Soils. 70 pp. 1908. (*Bulletin No. 50,* U. S. Dept. of Agriculture, Soils Bureau.) Discusses penetration and cohesion of soils, apparent specific gravity, or volume, moisture distribution and its various effects. Gives methods and results of experimental work.

CARLIN, J. P. Progress of Work at the United States Naval Academy. 3 ill. 1901. (*Engineering Record,* v. 43, pp. 449-452.) Gives results of tests on bearing power of piles made at Annapolis, Md.

CARTAULT. Note sur les Glissements de Terrains dans les Tranchées Argileuses de la Ligne de Paris à Lyon entre Brunoy et Bois-le-Roi. 1894. (*Annales des Ponts et Chaussées,* Memoires, ser. 7, v. 8, pp. 377-392.) Discusses treatment of landslides in clayey trenches along line of the Paris and Lyons Railway.

CARTER, FRANK H. Bracing and Sheeting Trenches. 1910. (*Engineering-Contracting,* v. 34, pp. 76-78.) Computes pressures on bracing and shoring for well under-drained excavations in virgin soil.

CARTER, FRANK H. Comparative Sections of Thirty Retaining Walls, and Some Notes on Retaining Wall Design. 1910. (*Engineering News,* v. 64, pp. 106-108.) Discusses theoretical earth pressures, giving formulas.

CARTER, HENRY H. Settlement of the Embankment Between Squantum and Moon Island, Boston Main Drainage Works. 1892. (*Journal,* Assoc. Eng. Soc., v. 11, pp. 355-362.) Gives results of time observations of settlement of embankment of mud, overlaid by gravel and filling material.

CARTWRIGHT, H. H. Tests of Grouting Gravel in River Beds. 1913. (*Engineering News,* v. 69, pp. 979-984.) Experiments indicated formation of hard foundation strata to be feasible, by forcing cement grout into soft foundation gravel.

CLARKE, D. D. A Phenomenal Land Slide. 1904. (*Transactions,* Am. Soc. C. E., v. 53, pp. 322-397.) Discussion, pp. 398-412. Describes surveys and explorations, covering a long period of time of slopes of two reservoirs of the City of Portland, Ore., to determine dimensions of landslide, its cause, and means for prevention.

CLEGG, SAMUEL. On Foundations, Natural and Artificial. 1851. (*Minutes of Proceedings,* Inst. C. E., v. 10, pp. 317-320.) Touches on bearing power of various soils.

COFFEY, GEORGE NELSON. Study of the Soils of the United States. 144 pp. 1912. (*Bulletin No. 85,* U. S. Dept. of Agriculture, Soils Bureau.) Bibliography, pp. 100-114. Discusses nature and origin of soils, soil-forming agencies, classification of soils; takes up the most important soils in detail, and gives their mineralogical composition.

COLBERG, OTTO. Eine Probelastung mit dem Betonpfahl-grundungs-system Strauss. 1909. (*Beton und Eisen,* v. 8, pp. 54-58.) Describes "Strauss" system and tests in very soft yielding soils.

COLE, HOWARD J. Concrete Piles. 12 ill., 1 dr. 1909. (*Transactions,* Am. Soc. C. E., v. 65, pp. 467-487.) Discussion, 3 diag., 4 ill., 5 dr., pp. 488-513. Important discussion on concrete piles. Formulas for their bearing power are derived.

COMPOSITE SAND AND ROCK FOUNDATIONS FOR A TALL BUILDING. 2 dr. 1910. (*Engineering News,* v. 63, pp. 24-26.) Describes foundations, base of Municipal Building of New York City, 25-story office building with a tower rising 560 ft. above street level. Decision is for two-thirds of building to be founded on rock, the remainder on sand.

CONCRETE FOUNDATIONS IN SHIFTING GROUND. 1908. (*Engineering News,* v. 59, p. 573.) Stable foundations were obtained by founding chain conveyor footings on rock below earth which had been gradually creeping or sliding.

CONSIDERE. Note sur la Poussee des Terres. 1870. (*Annales des Ponts et Chaussées,* ser. 4, v. 19, pp. 547-594.) Extension of Mr. Levy's theory on earth pressure. See *Comptes Rendus Hebdomadaires des Seances* de l'Academie des Sciences, v. 68, p. 1456.

CONSTABLE, CASIMIR. Retaining Walls.—An Attempt to Reconcile Theory with Practice. 3 diag. 1874. (*Transactions,* Am. Soc. C. E., v. 3, pp. 67-75.) Gives results of a number of toy experiments with wooden bricks and peas for filling.

CONSTRUCTING THE FOUNDATIONS OF THE SEAMEN'S CHURCH INSTITUTE, New York. 1912. (*Engineering Record,* v. 65, pp. 105-107.) Pneumatic wall caissons were sunk through mud, clay, sand, and hardpan, to about 40 ft. below street.

COOPER, ROBERT ELLIOTT. Causes of Earth Slips in the Slopes of Cuttings and Embankments of Railways, and How to Prevent or Remedy Them. 1899. (*Minutes of Proceedings,* Inst. C. E., v. 138, pp. 383-385.)

——1899. (*The Engineer,* London, v. 87, p. 612.)

——1899. (*Engineering,* v. 67, p. 826.) States that "slips in embankments" are chiefly of two descriptions. First, where the material composing the embankment slips. Secondly, where the surface of the ground on which the embankment rests slips away on the underlying strata. Gives remedy.

CORNISH, VAUGHAN. Panama Canal and the Philosophy of Landslides. 1913. (*Edinburgh Review,* v. 217, pp. 21-42.) Mainly deals with description and occurrence of landslides along the Panama Canal.

CORTHELL, ELMER LAWRENCE. Allowable Pressures on Deep Foundations. 98 pp. 1907. Bibliography, pp. 37-40.

——Abstract. (*Minutes of Proceedings,* Inst. C. E., v. 165, pp. 249-251.) See also book note, *Engineering Record,* v. 56, p. 161. Paper before the Institution of Civil Engineers; contains a large amount of information and tabulated data showing allowable ranges of pressure of structures on fine sand, coarse sand and gravel, sand and clay, alluvium and silt, hard clay, and hardpan. Contains also letters from various engineers giving information from projects under personal observation of writers, and abstracts of many technical papers on foundations.

——Short abstract. 1906. (*Engineering Record,* v. 54, p. 629.) See also editorial "Bearing Capacity of Earth Foundation Beds." *Engineering Record,* v. 54, pp. 647-648.

COTTERILL, GEORGE F. Supporting Power of Piles. 1902. (*Engineering Record*, v. 45, pp. 231-232.) Letter to editor giving results of tests on piles in Seattle.

COWLES, WALTER L. Lateral Pressure in Clay from Superimposed Loads. 1912. (*Journal*, Western Society of Engineers, v. 17, pp. 746-750.) Deduces formula to serve as basis for determining horizontal pressure from a vertical, superimposed load.

CRUTTWELL, GEORGE EDWARD WILSON. Foundations of the River Piers of the Tower Bridge. With discussion. 1893. (*Minutes of Proceedings*, Inst. C. E., v. 113, pp. 117-150.) Gives reasons for limiting pressure on the London clay to 4 tons per sq. in., pp. 148-149.

CUNNINGHAM, BRYSSON. Dock and Harbour Engineer's Reference Book. 1914. 319 pp. Data on foundations for quay and dock walls, earth pressure, angles of repose, and bearing powers of different soil materials, pp. 145-152.

CURTIS, W. G. Notes on a Mountain Slide. 1 diag., 1 dr., 3 ill. 1891. (*Transactions*, Am. Soc. C. E., v. 24, pp. 556-563.) With discussion. In Northern California.

DARWIN, GEORGE HOWARD. On the Horizontal Thrust of a Mass of Sand. 10 diag., 2 dr. 1882. (*Minutes of Proceedings*, Inst. C. E., v. 71, pp. 350-378.) See also note by Gaudard. Attempts to verify the theoretical investigations of Concomb, Rankine, and others by a series of experiments.

DAWKINS, BOYD. On the Relation of Geology to Engineering. 2 diag., 2 ill. 1898. (*Minutes of Proceedings*, Inst. C. E., v. 134, pp. 254-277.) "Relation of superficial accumulation to solid rocks," pp. 269-272.

DAWLEY, W. M. Drainage of Soft Spots in Old Roadbed. 1907. (*Proceedings*, Eighth Annual Convention, Am. Ry. Eng. and Maintenance of Way Assoc., pp. 541-554.) Object of drainage is to prevent settlement of track at soft spots.

De BURGH, ERNEST MACARTNEY. Pile Sinking by Means of a Hydraulic Jet at Moruya and Carrington Bridges, New South Wales. 1 pl. 1902. (*Minutes of Proceedings*, Inst. C. E., v. 150, pp. 340-351.) Tests of sustaining power of piles, p. 348.

DESNOYER, CROIZETTE. Mémoire sur l'Etablissement des Travaux dans les Terrains Vaseux de Bretagne. 8 pl. 1864. (*Annales des Ponts et Chaussées*, ser. 4, v. 7, pp. 275-396). French foundation practice on alluvial soils.

DEVELOPMENT OF SHALLOW AND DEEP FOUNDATIONS FOR CHICAGO BUILDings. 1904. (*Engineering News*, v. 52, pp. 560-563.) Gives brief information as to nature of Chicago soils at different depths.

DIAGRAM FOR OVERTURNING MOMENTS ON RETAINING WALLS FOR EARTH or water. 1907. (*Engineering News*, v. 57, p. 460.) Diagram was constructed by Charles H. Hoyt.

DILLEY, WILFRID JOSEPH. Footings in Foundations. 6 diag. 1905. (*Minutes of Proceedings*, Inst. C. E., v. 163, pp. 309-318.) Discusses stresses and reactions when center of pressure coincides with geometrical center of area of foundation.

DRAINING AND STEADYING SLIPS. 1904. (*Railroad Gazette*, v. 37, pp. 317-318.)

DUNCAN, LINDSAY. Plumbing a Leaning Retaining Wall and Bridge Abutment. 1906. (*Engineering News*, v. 55, p. 386.)

EARTH SLIDE, AT THE NORTH DIKE OF THE WACHUSETT RESERVOIR, 1907. (*Engineering Record*, v. 55, pp. 515-516.) Soil of embankment was a very fine impermeable sand.

EARTHWORKS. 1909. (*The Engineer*, London, v. 107, p. 265.) Considers especially angle of slope desirable, and drainage.

ECCENTRIC LOAD ON PILES OR RIVETS. 1912. (*Engineering News*, v. 67, p. 1190.) Note: Giving formula and a table of constants.

ECKARDT, A. Die mechanischen Einwirkungen des Abbaues auf des Verhalten des Gebirges. 1913. (*Glückauf*, v. 49, pt. 1, pp. 352-361, 397-403.) Studies reaction between mine workings and earth pressure, mine roofs being held up by arch action in the overlying masses.

EISENWERTH, ADOLPH SCHMOLL V. Mitteilungen en über pneumatische Fundirungen und Erfahrungsresultate über die dabei vorkommenden Reibungswiderstande. 1877. (*Zeitschrift des Vereines deutscher Ingenieure*, v. 21, pp. 433-456.) Gives experimental results on frictional resistance encountered by caissons at various depth in various strata, and methods of calculation of the resistance of friction.

——Abstract. 1878. (*Minutes of Proceedings*, Inst. C. E., v. 52, pp. 298-302.)

ELLIS, G. W. New Pile Formula Desired. 1905. (*Engineering Record*, v. 52, p. 390.) Letter to editor criticizing the accepted formulas for bearing power of piles, and inviting the engineering profession to evolve a new one.

EMPERGER, FRITZ von. Probelastung einer "Compressol" Pylone. 1908. (*Beton und Eisen*, v. 7, pp. 49-55.) Gives tabulated results of tests.

ESPITALLIER, G. La Fondation des Ouvrages en Terrains Vaseux. 1905. (*Le Génie Civil*, v. 47, pp. 329-331.) Considers difficult foundation work at Saigon, Indo China, on muddy ground.

——Abstract, 1906. (*Minutes of Proceedings*, Inst. C. E., v. 164, pp. 451-452.)

EVEREST, J. H. Treatise on Retaining Wall Design. 1911. (*The Canadian Engineer*, v. 21, pp. 192-193, 237, 264-265.) Considers earth pressure, slope, weights of materials, etc.

EWEN, JOHN M. Foundations for Chicago Buildings. 1905. (*Journal*, West. Soc. of Engrs., v. 10, pp. 687-704.) Gives information regarding nature of Chicago soils at different depths, and foundation practice.

FARGO, WILLIAM G. Experience with Steel Sheet-piling in Hard Soils. 1907. (*Engineering News*, v. 57, pp. 374-375.) Paper before Michigan Engineering Society. Methods and costs. Clay hardpan soil was encountered.

——1907. (*Engineering and Contracting*, v. 27, pp. 193-195.)

——Condensed. 1907. (*Engineering Record*, v. 55, pp. 175-176.)

FLAMANT, A. Note sur la Poussee des Terres, 1882. (*Annales des Ponts et Chaussées*, ser. 6, v. 3, pp. 616-624.) Mostly a review of Mr. Baker's paper in *Minutes of Proceedings*, Inst. C. E., v. 65, p. 140.

FORCHHEIMER, PH. Ueber Sanddruck und Bewegungserscheinungen im innern trockenen Sandes. 7 diag., 1 pl. 1883. (*Zeitschrift* des Oesterreichischen Ingenieur und Architekten Vereines, v. 34, pp. 111-126.) Gives a brief historical review of the work done on sand pressure, lateral movement of sand and angle of repose of sand masses. Describes his own experiments giving the kinds of sand used, apparatus, methods of measurement, and results obtained.

FORCHHEIMER, PH. Ueber Sanddruck und Bewegungserscheinungen im innern trockenen Sandes. 6 diag., 1 pl. 1883. (*Zeitschrift* des Oesterreichischen Ingenieur und Architekten Vereines, v. 35, pp. 103-109.) Further experiments with various positions of the experimental retaining wall and the sand mass and various conditions.

FORD, FREDERICK L. Settlement of Lorraine Street, Hartford, Conn. 1902. (*Engineering Record*, v. 45, pp. 172-173.) Describes progress of slow landslide and drainage method used in preventing further damage.

FORMULAS FOR SAFE LOADS OF BEARING PILES. 1888. (*Engineering News*, v. 20, pp. 509-512.) Editorial discussion of John C. Trautwine's views on the subject and a comparison of latter's formula with various other formulas.

FOUNDATION FOR THE CANAL AND CLAIBORNE ELECTRIC RAILWAY POWER House, New Orleans, La. 1 dr. 1897. (*Engineering News*, v. 38, pp. 124-125.) Foundation soil is new material, depth of about 35 ft., underlaid with natural stratum of gravel and shells, 3 to 7 ft. thick. Building walls were carried on piles.

FOUNDATION LOADS. 1911. (*The Builder*, London, v. 101, pp. 332-333, 453-454, 780-781.) Gives figures to show pressures of structures on different earth soils at which no settlement was observed. Includes also compressive resistance of different stones, and discusses foundations under differing soil conditions.

FOUNDATIONS FOR THE NEW CITY HALL IN CHICAGO. 2 ill. 1909. (*Engineering Record*, v. 59, pp. 745-747.) Bed-rock at site of building is overlaid by successive layers of water-bearing strata, hardpan, blue clay, yellow clay, muck, and refuse filling material.

FOUNDATIONS OF MUD FLOTATION. 1905. (*Engineering Record*, v. 52, p. 251.) Editorial indicating methods of increasing the bearing power of mud.

FOUNDATIONS OF THE FIREMEN'S INSURANCE BUILDING, IN NEWARK. 1911. (*Engineering Record*, v. 64, pp. 636-637.) Excavations were through quaking sand and a subterranean stream. Steel sheet-piling was used.

FOUNDATIONS OF THE NEW POST OFFICE AND GOVERNMENT BUILDING AT Chicago. 5 diag., 1 dr. 1898. (*Engineering News*, v. 39, pp. 66-68.) Indicates successive soil strata for four borings, showing in general blue clay of varying consistency between layer of filling and hardpan.

FOUNDATIONS OF THE NORTHWESTERN RAILWAY TERMINAL, CHICAGO. 1909. (*Engineering Record*, v. 59, pp. 595-597.) Details of unusual foundation work through soft soil, blue clay, and hardpan to rock.

FOWLER, CHARLES EVAN. Practical Treatise on Subaqueous Foundations, Including the Coffer-dam Process for Piers, and Dredges and Dredging, with Numerous Practical Examples from Actual Work. ed. 3. 814 pp. 1914. John Wiley and Sons, New York. Bearing capacity of soils, pp. 409-423, 491.

FRANCIS, GEORGE B. Foundations. 1 dr. 1903. (*Journal,* Assoc. Eng. Soc., v. 30, pp. 336-342.) Gives author's figures for safe supporting loads for ledge rock, hardpan, gravel, clean sand, dry clay, wet clay, and loam.

FREE, E. E. Movement of Soil Material by the Wind. 272 pp. 1911. (*Bulletin No. 68,* U. S.-Dept. of Agriculture, Soils Bureau.) "Bibliography of Eolian Geology," by S. C. Stuntz and E. E. Free, pp. 172-272. A very thorough manuscript on the subject, giving many references to original sources. Treats on translocating agents in general, on mechanics of wind translocation, drifting sand and sand dunes, dust storm and dust falls, etc.

GAILLARD, D. D. Culebra Cut and the Problem of the Slides. 1912. (*Scientific American,* v. 121, pp. 388-390.)

GARDNER, FRANK D. Electrical Method of Moisture Determination in Soils. 24 pp. 1898. (*Bulletin No. 12,* U. S. Soils Division.)

GAUDARD, JULES. On Foundations. 1877. (*Minutes of Proceedings,* Inst. C. E., v. 50, pp. 112-147.) Gives various methods of foundation construction, safe loads for various soils, and information on friction of cylinders through the soil.

GAUDARD, JULES. Note on Mr. J. H. Darwin's Paper "On the Horizontal Thrust of a Mass of Sand." 1882. (*Minutes of Proceedings,* Inst. C. E., v. 72, pp. 272-274.)

GIBSON, THOMAS. Huelva Pier of the Rio Tinto Railway. With discussion. 2 dr. 1878. (*Minutes of Proceedings,* Inst. C. E., v. 53, pp. 130-163.) Gives supporting power of mud, pp. 135-137; tables, pp. 144-158.

GODFREY, EDWARD. Design of Reinforced Concrete Retaining Walls. 1906. (*Engineering News,* v. 56, pp. 402-403.) Considers lateral pressure of different materials, angles of repose, and necessary calculations.

GOODRICH, ERNEST P. Lateral Earth Pressures and Related Phenomena. 44 diag., 3 dr., 1 ill. 1904. (*Transactions,* Am. Soc. C. E., v. 53, pp. 272-304.) Discussion, pp. 305-321. Experimentally determines ratio of lateral to vertical pressure. Gives series of conclusions. See also editorial "Lateral Earth Pressure," *Engineering Record,* v. 49, pp. 633-634.

——Abstract. 1904. (*Minutes of Proceedings,* Inst. C. E., v. 158, pp. 450-451.)

GOODRICH, ERNEST P. The Supporting Power of Piles. 18 diag., 3 dr. 1902. (*Transactions,* Am. Soc. C. E., v. 48, pp. 180-212.) Discussion, pp. 213-219. Author develops a general formula for the bearing power of piles and compares it with numerous existing formulas.

——Abstract. (*Minutes of Proceedings,* Inst. C. E., v. 149, pp. 383-384.) See also editorial, "Supporting Power of Piles," *Engineering Record,* v. 45, p. 97. See also letter to editor, 1 diag., 1902, *Engineering Record,* v. 45, p. 184.

GOODRICH, ERNEST P. Supporting Power of Piles. 1910. (*Proceedings,* Am. Ry. Eng. and Maintenance of Way Assoc., v. 11, pt. 1, pp. 217-226.) See also editorial, *Engineering Record,* v. 61, p. 744. Suggests further experimental work for developing a dependable formula for supporting power of piles.

GOW, CHARLES R. Concrete Piles, 1 dr., 3 ill. 1907. (*Journal,* Assoc. Eng. Soc., v. 39, pp. 255-265.) Gives tests on supporting power of concrete piles.

——Abstract. 1907. (*Engineering News,* v. 59, pp. 305-307.)

GRAFF, C. F. High Reinforced Concrete Retaining Wall Construction at Seattle, Wash. 1905. (*Engineering News,* v. 53, pp. 262-264.)

GREGORY, CHARLES HUTTON. On Railway Cuttings and Embankments, with an Account of Some Slips in the London Clay, on the Line of the London and Croydon Railway. 1844. (*Minutes of Proceedings,* Inst. C. E., v. 3, pp. 135-145.) Discussion, pp. 145-173. Describes slips in railway cuttings caused by increased lateral pressure due to vibrations.

GRIFFITH, JOHN H. The Ultimate Load on Pile Foundations a Static Theory. 6 diag. 1910. (*Transactions,* Am. Soc. C. E., v. 70, pp. 412-442.) Discussion, pp. 442-447. Attempts to construct a formula for safe bearing load of piles by statical methods. See also editorial in *Engineering Record,* v. 61, p. 744.

GUARINI, EMILE. Mechanical Compression of the Ground in the Construction of Foundations. 1906. (*Scientific American,* v. 108, pp. 476-477.) Compression is by treatment of ground with concrete in such manner as to compress it laterally and depthwise.

HANCOCK, EDWIN. Bearing Power of Moist Blue Clay, Result of Tests in the Loop District, Chicago, Ill. 1912. (*Journal,* Western Soc. of Engrs., v. 17, p. 745.)

——1913. (*Engineering News,* v. 69, p. 464.)

HANCOCK, WALTER C. Physical Properties of Clays. 1913. (*Journal,* Royal Soc. of Arts, v. 61, pp. 560-567.) Careful study and clear exposition of clays, without special reference to their value as foundation material.

HARRISON, THOMAS ELLIOT. On the Tyne Docks at South Shields: and the Mode Adopted for Shipping Coals. 1859. (*Minutes of Proceedings,* Inst. C. E., v. 18, pp. 490-503.) Tells of experiments made on bearing power of mud or slate, p. 493.

HASELER, E. Erddruck nach der Lehre vom Erdprisma des grossten Druckes. 13 diag. 1905. ("Handbuch der Ingenieurwissenschaften", 4 Aufg., I Teil, 2 Band, pp. 294-314.) Literatur, p. 403. Gives mathematical theory of earth pressure.

HASWELL, CHARLES H. Pile Driving Formulas: Their Construction and Factors of Safety, 1899. (*Transactions,* Am. Soc. C. E., vol. 42, pp. 267-276.) Discussion, 1 diag., pp. 277-287.

HASWELL, CHARLES H. On Formulas for Pile-Driving. 1893. (*Minutes of Proceedings,* Inst. C. E., v. 115, pp. 315-320.) Discusses formulas of Rankine, Sanders, Molesworth, and others, and then derives his own formula.

HATCH, JAMES N. Chicago Foundations. 1 diag., 3 dr. 1904. (*Architects' and Builders' Magazine,* v. 5, pp. 555-561; v. 6, pp. 18-24.) Shows different strata of soil found, with depths, from borings at time of construction of Post Office Building. Discusses generally soils underlying business district of Chicago.

HAYTER, HARRISON. Charing Cross Bridge. 1863. (*Minutes of Proceedings,* Inst. C. E., v. 22, pp. 512-539.) Gives some information on frictional resistance of bed to descent of cylindrical piers, pp. 514-515.

HENDRY, M. C. Bearing Power of Piles. 1909. (*The Canadian Engineer,* v. 16, pp. 485-490.) Discusses formulas, and their basis, action of soil, etc.

HERING, RUDOLPH, comp. Bearing Piles. (*Engineering News,* v. 5, pp. 56, 88, 103-104, 116, 124.) Gives formulas for their sustaining power, size, and disposition in any foundation.

HILGARD, EUG. W. On the Silt Analysis of Soils and Clays. 2 ill. 1873. (*Proceedings,* Am. Assoc. for Advancement of Science, v. 22, pp. 54-70.) Describes his method of mechanical analysis of soils, and gives results of extensive experiments, mostly on soils of Mississippi.

——1873. (*American Journal of Science and Arts,* v. 106, pp. 288-296, 333-339.)

HILGARD, E. W. Soils, their Formation, Properties, Composition, and Relations to Climate and Plant Growth in the Humid and Arid Regions. 593 pp. 1906. Macmillan Company, New York. Contents: Origin and formation of soils, physics of soils, soils and native vegetation.

HILL, GEORGE. Foundations. 1893. (*American Architect,* v. 41, pp. 179-182.) Discusses testing of soils, bearing value of various soils, bearing value of piles, and treatment of soils, etc.

HIRSCHTHAL, M. Some Contradictory Retaining Wall Results. 1 diag. 1912. (*Engineering News,* v. 67, pp. 799-800.) Letter to editor reviewing some accepted formulas of earth pressure on retaining walls.

HISELY, Constructions Diversés pour Determiner la Poussee des Terres, sur un Mur de Soutenement. 1899. (*Annales des Ponts et Chaussées,* ser. 7, v. 17, mémoires, pp. 99-120.) Develops a general graphical solution applicable to any character of load.

HOOPER, HENRY. Description of the Pier at Southport, Lancashire. 1861. (*Minutes of Proceedings,* Inst. C. E., v. 20, pp. 292-299.) Gives load used on cast-iron piles, p. 295.

HOWE, ERNEST. Landslides in the San Juan Mountains, Colorado, Including a Consideration of Their Causes and Their Classification. 55 pp. 1909. (U. S. Geol. Survey, *Professional Paper 67.*)

HOWE, HORACE J. Piles and Pile-driving, New and Old. 1898. (*Journal,* Assoc. Eng. Soc, v. 20, pp. 257-294.) Discussion, 1 ill., pp. 294-312. Reviews a number of important papers on piles and pile-driving.

HOWE, MALVERD A. Foundations: A Short Text-Book on Ordinary Foundations, including a Brief Description of the Methods Used for Difficult Foundations. 110 pp. 1914. John Wiley & Sons, New York. References at the end of each chapter. Supporting capacity of soils, pp. 1-12.

HUNT, RANDELL. Principles Governing the Design of Foundations for Tall Buildings. 1896. (*Journal,* Assoc. Eng. Soc., v. 17, pp. 1-26.) Gives brief information and suggestions regarding soil foundations and the desirable factor of safety in loading soils.

HUNT, RANDELL. Supporting Power of Soils. 1888. (*Journal,* Assoc. Eng. Soc., v. 7, pp. 189-196.) Gives many examples of foundations on various soils.

——Abstract. 1888. (*Minutes of Proceedings,* Inst. C. E., v. 94, pp. 327-328.)

HURTZIG, ARTHUR CAMERON. Note on the Friction of Timber Piles in Clay. 1 dr., 1 pl. 1881. (*Minutes of Proceedings,* Inst. C. E., v. 64, pp. 311-315.) Also deduces a formula for extreme load that could be carried by a pile.

HUTTON, W. R. Foundations of High Buildings. 1893. (*Engineering Record,* v. 28, pp. 268-270.) Paper before Congress of Architects, Chicago. Discusses New York and Chicago foundation practice, and the nature of the underlying soils. Considers extent of lateral displacement of soil due to vertical load.

IMPROVEMENT OF A SIDING CUT ON THE CLEVELAND, CINCINNATI, CHICAGO and St. Louis Railroad. 1908. (*Engineering News,* v. 59, pp. 478-479.) Improved drainage made the slide less active.

ISAACS, JOHN D. Stopping a Troublesome Slide at a Summit Tunnel. 1895. (*Journal,* Assoc. Eng. Soc., v. 15, pp. 113-123.) Concrete retaining wall was constructed, pierced by a tunnel.

JENSEN, J. NORMAN. Hardpan and Other Soil Tests. 1913. (*Engineering News.* v. 69, pp. 460-463.) Tests of various Chicago soils.

K, F. P. Short Formula for Bearing Piles. 1888. (*Engineering News,* v. 20, p. 326.) A letter to editor signed F. P. K.

KAFKA, RICHARD. Praktische Anwendungen der Methoden zur Bestimmung der Zulassigen Pfahlbelastung. 1909. (*Beton und Eisen,* v. 8, pp. 161-164, 196-198, 212-216.) Static, geometrical method is used in calculating.

KIRK, P. R. Graphic Methods of Determining the Pressure of Earth on Retaining Walls. 1899. (*The Builder,* London, v. 77, pp. 233-235.)

KLEIN, ALBERT. Die Form der Winkelstutzmauern aus Eisenbeton mit Rucksicht auf Bodendruck und Reibung in der Fundamentfuge. 1909. (*Beton und Eisen,* v. 8, pp. 384-387.)

KOENIG, ARNOLD C. Dams on Sand Foundations: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling. 4 dr. 1911. (*Transactions,* Am. Soc. C. E., v. 73, pp. 175-189.) Discussion, 2 diag., 6 dr., pp. 190-224.

KREUTER, FRANZ. New Method for Determining the Supporting Power of Piles. 1896. (*Minutes of Proceedings,* Inst. C. E., v. 124, pp. 373-377.) Analytical method verified experimentally. See also editorial "A New Formula for the Supporting Power of Piles," *Engineering Record,* v. 33, p. 343.

——Abstract. 1896. (*Engineering Record,* v. 33, p. 330.)

——Condensed. 1896. (*Railway Review,* v. 36, p. 262.)

KRIEGSMAN, EUGEN F. Pile-Driver Diagram. 1 diag. 1912. (*Engineering Record,* v. 65, p. 417.) Gives logarithmic diagram for ascertaining the bearing power of piles by measuring the penetration under the fall of a drop-hammer.

KURDJUMOFF, V. J. Foundations on Natural Ground. 1893. (*Minutes of Proceedings,* Inst. C. E., v. 111, pp. 414-415.) Abstract from *Der Civilingenieur,* 1892, p. 293. Deals with foundations in loose or sandy soils. Uses photographic methods. Finds that Rankine's formula, based on the theory of granular masses, does not give accurate results.

LAGRENE, H. de. Note sur la Poussee des Terres Avec ou Sans Surcharges. 8 diag., 2 dr. 1881. (*Annales des Ponts et Chaussées,* ser. 6, v. 2, pp. 441-471.) Gives calculations for earth pressure of level surfaces on vertical retaining walls.

——Abstract. 1882. (*Minutes of Proceedings,* Inst. C. E., v. 68, pp. 336-337.)

LANDRETH, WILLIAM B. The Improvement of a Portion of the Jordan Level of the Erie Canal. With discussion. 1900. (*Transactions,* Am. Soc. C. E., v. 43, pp. 566-602.) Paper and discussion give valuable information on quicksand and its bearing power.

——Abstract. 1900. (*Engineering Record,* v. 41, p. 137.)

LARGE CONCRETE PILE INSTALLATION; Data on Preliminary Bearing Tests and Details of Driving more than 11 000 Cast-in-Place Piles for Steel Plant Foundations. 1913. (*Engineering Record,* v. 67, pp. 36-37.) Piles were driven through layers of granulated slag, loam, yellow clay, sand, sand and clay, and running sand, to rock.

LEA, SAMUEL H. Foundation Construction for the New State Capitol of South Dakota. 2 dr., 1 ill. 1908. (*Engineering Record,* v. 57, pp. 437-438.) Tests were made of bearing capacity of hard, firm soil a few feet below surface.

LEFEBVRE, RENE. Mémoire sur la Constitution des Terres et sur les Accidents dans les Terrains Argileaux. 6 dr. 1878. (*Annales des Ponts et Chaussées*, ser. 5, v. 16, pp. 390-445.) Paper was translated by Lieut. F. A. Mahan, and considers slips in cuts and fills as due to pressure of contained water and dependent on the permeability and penetrability of the earths present. Studies the construction of various earths in this respect. Develops a theory of slips, and discusses practical methods of prevention, giving examples of their application.

——Translated. 6 dr. 1882. (*Transactions*, Engrs. Soc. of Western Pennsylvania, v. 1, pp. 70-105.)

LENTZ, H. Drawing Piles in Cuxhaven Harbour. 1880. (*Minutes of Proceedings*, Inst. C. E., v. 59, pp. 396-397.) Discusses the difficult operation of drawing piles, and gives the pressures required in Cuxhaven case.

——Abstract from *Deutsche Bauzeitung*, 1879, p. 340.

LEONARD, HUGH. Foundations for Buildings in Calcutta. 4 dr. 1895. (*Journal*, Royal Inst. of British Architects, v. 44, pp. 593-595.) Results of tests of sandy clay.

——(*Engineering*, v. 20, p. 103.)

——Abstract. 4 dr. 1895. (*Engineering Record*, v. 32, p. 31.)

LEONARD, HUGH. Supporting Power of Rocks and Soils; Experiments at Calcutta. 1900. (*Journal*, Royal Inst. of British Architects, v. 49, pp. 390-393.) Tabulated data from results of experiments, showing supporting power of ordinary, undisturbed, alluvial soil of Calcutta. Tests were made to depth of 11 ft. where the successive formations were rich soil, dry bluish clay, dry brownish clay, brownish clay mixed with clay and more moist, in pure peat, and wet sand and clay.

LESLIE, BRADFORD. Account of the Bridge over the Gorai River on the Goalundo Extension of the Eastern Bengal Railway. With discussion. 1872. (*Minutes of Proceedings*, Inst. C. E., v. 34, pp. 1-42.) In discussion Major J. Browne tells of friction of brick wells in clay, pp. 34-35.

LEVY, MAURICE. Essai sur une Theorie Rationnelle de l'Equilibre des Terres Fraichement Remueés et ses Applications au Calcul de la Stabilité des Murs de Soutenement. 1869. (*Comptes Rendus Hebdomadaires des Seances* de l'Academie des Sciences, v. 68, pp. 1456-1458.) Develops a theory of earth pressures, and shows its application in design of retaining walls.

LEYGUE, L. Nouvelle Recherche sur la Poussee des Terres et le Profil de Revêtement le Plus Economique. 1885. (*Annales des Ponts et Chaussées*, Mémoires, ser. 6, v. 10, pp. 788-1003.) Contains results of extended experiments on cohesion of earth, direction, point of application, and amount of earth pressure, and calculations and formulas for retaining walls, etc.

McALPINE, WILLIAM J. The Foundations of the New Capitol at Albany, N. Y. 1874. (*Transactions*, Am. Soc. C. E., v. 2, pp. 287-288.) Briefly discusses bearing power of soils, and gives results of tests made.

McALPINE, WILLIAM JARVIS. Supporting Power of Piles; and on the Pneumatic Process for Sinking Iron Columns, as Practised in America. With discussion. 1 pl., 2 dr. 1868. (*Minutes of Proceedings*, Inst. C. E., v. 27, pp. 275-319.) Discusses formulas of Sander, Molesworth, and Weisbach, and deduces his own relation based on extensive experiments.

McALPINE, WILLIAM JARVIS. Foundation of the New Capitol at Albany, N. Y. 1879. (*Minutes of Proceedings*, Inst. C. E., v. 57, pp. 198-207.) Describes experiments, and also various considerations taken into account, for determining the bearing power of soil at Albany, N. Y.

MacDONALD, DONALD F. Landslides of Culebra Cut. 1912. (Annual Report, Isthmian Canal Commission, 1912, pp. 205-214.) Names four types of slides, (1) those resulting from structural breaks and deformations, (2) normal or gravity slides, (3) fault-zone slides, (4) those resulting from weathering and surface erosion. Causes and remedies for each are discussed.

MacDONALD, DONALD F. Slides in the Culebra Cut at Panama; a Review of Geological Conditions in the Canal Site, Together with a Description of the Types of Slides and Their Causes. 1912. (*Engineering Record*, v. 66, pp. 228-233.) See also editorial, p. 225. Concludes that "when the slopes shall have been reduced to the proper angle * * *, the slide problem will be practically solved."

MacDONALD, DONALD F. Sliding Ground in Culebra Cut. 1913. (*Engineering News*, v. 70, p. 408.) Gives reasons why methods of preventing earth and rock slides proposed by Rice would not be applicable to Culebra Cut slides, along the Panama Canal.

McGEE, WILLIAM J. Soil Erosion. 33 pl. 60 pp. 1911. (*Bulletin No. 71*, U. S. Dept. of Agriculture, Soils Bureau.) Written primarily from the agriculturist's standpoint. Discusses the agencies producing soil erosion.

MACONCHY, G. C. Earth Pressures on Retaining Walls. 1898. (*Engineering*, v. 66, pp. 256-257, 484-485, 641-643.) Gives simpler method for calculating overturning moments.

MAGINNIS, OWEN B. Consideration of the Earth's Surface in its Relation to Building Construction. 1907. (*Architects and Builders Magazine*, v. 9, pp. 82-84.) General discussion of value of different soil materials in foundation work, and their bearing capacities. Very little quantitative information given.

MAIN, J. A. Graphic Determination of Pressures on Retaining Walls. 1912. (*The Engineer*, London, v. 113, p. 220.)

MARBUT, CURTIS F., and others. Soils of the United States. 791 pp. 1913. (*Bulletin No. 96*, U. S. Dept. of Agriculture, Soils Bureau.) Virtually a complete handbook on the subject.

MAYER, R. Apparatus for Determining the Safe Load upon Foundations. 1900. (*Minutes of Proceedings*, Inst. C. E., v. 142, pp. 408-409.) Abstract of paper in *Schweizerische Bauzeitung*, v. 28, No. 22.

MAYNARD, HENRY NATHAN. New Ross Bridge. With discussion. 1871. (*Minutes of Proceedings*, Inst. C. E., v. 32, pp. 146-191.) Discussion refers to Sander's formula for supporting power of piles, pp. 165-167.

MEEM, J. C. The Bracing of Trenches and Tunnels, with Practical Formulas for Earth Pressures. 2 diag., 5 ill., 13 dr. 1908. (*Transactions*, Am. Soc. C. E., v. 60, pp. 1-23.) Discussion, 5 ill., 10 diag., 54 dr., pp. 24-100. Develops a theory of earth pressure and on basis of this theory deduces analytical relations.

——Abstract. 1908. (*Minutes of Proceedings*, Inst. C. E., v. 171, pp. 435-436.)

——Abstract. 1 ill., 3 dr. 1907. (*Engineering Record*, v. 56, pp. 494-496.) See also editorial "Sheet-piling and Earth Pressure." *Engineering Record*, v. 56, p. 528.

MEEM, JAMES C. Notes and Experiments on Earth Pressures. 1912. (*Proceedings*, Engrs. Club of Philadelphia, v. 29, pp. 114-130.) Includes calculations, based mainly on experiments carried out on laboratory scale.

MEEM, J. C. Pressure, Resistance, and Stability of Earth. 2 diag., 12 ill., 11 dr. 1910. (*Transactions*, Am. Soc. C. E., v. 70, pp. 352-388.) Discussion, pp. 389-411. Gives results of practical experiments, makes deductions and gives applications of the deductions to works carried out.

——Abstract. 1910. (*Minutes of Proceedings*, Inst. C. E., v. 182, p. 355.) See also editorial in *Engineering Record*, v. 61, p. 744.

MERRICK, A. W. Clay Slide at the Boone Viaduct, Boone, Iowa. 1906. (*Journal*, Western Soc. of Engrs., v. 11, pp. 332-334.) Discussion, pp. 335-339. Successful draining cured the slide.

——1907. (*Proceedings*, Eighth Annual Convention, Am. Ry. Eng. and Maintenance of Way Assoc., pp. 555-582.)

MERRIMAN, MANSFIELD. On the Theories of the Lateral Pressure of Sand Against Retaining Walls. 1887 (*School of Mines Quarterly*, v. 9, pp. 109-112.) Paper read before the American Association for the Advancement of Science. Reviews accepted formulas, and claims that they are based but little on experiments.

——Abstract. 1888. (*Engineering News*, v. 19, p. 152.)

——Abstract. 1887. (*Proceedings*, Am. Assoc. for Advancement of Science, v. 36, p. 166.)

MILLER, RUDOLPH P. Allowable Pressure on Hardpan. 1910. (*Engineering Record*, v. 62, p. 783.) Letter to editor.

MOFFET, J. S. D. Mistaken Ideas with Reference to the Resultant Force and the Maximum Pressure in Retaining Wall Calculations. 1903. (*Feilden's Magazine*, v. 9, pp. 197-199.) Illustrates simply and clearly the different forces entering.

MOHLER, CHARLES K. Earth Pressures. 1 ill., 10 diag. 1910. (*Journal*, Western Soc. of Engrs., v. 15, pp. 765-791.) Discussion, 5 ill., 13 diag., pp. 791-827.) See also editorial, *Engineering Record*, v. 61, p. 744. A study of the sliding prism theory of Vauban after the graphics of Rebhann and of the analytical theory of Rankine, showing lack of agreement and break-downs in the theories when worked out for results; also formulas and results from a new method.

MOHLER, C. K. Tables for the Determination of Earth Pressures on Retaining Walls. 1909. (*Engineering News*, v. 62, pp. 588-589.)

MOLESWORTH, SIR GUILFORD. Physical Properties of Black Cotton Soil. 5 diag. 1898. (*Minutes of Proceedings*, Inst. C. E., v. 132, pp. 209-213.) In discussion on W. L. Strange's paper: "Reservoirs with High Earthen Dams in Western India."

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TEST OF CONCRETE PILES. 1910. (*Engineering Record*, v. 62, p. 715.) Gives methods and results of tests of concrete piles.

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TESTING FOUNDATIONS AT THE MUNICIPAL BUILDING, NEW YORK. 3 dr., 1 ill. 1911. (*Engineering Record*, v. 63, pp. 196-197.) Gives results of loading tests with pier resting on sand, about 35 ft. below the surface and 80 ft. above rock level.

TESTING SOIL BELOW THE SURFACE FOR FOUNDATION LOADS. 1 000 w. 2 ill. 1910. (*Engineering Record*, v. 62, p. 71.) Tests were made during construction of warehouse building in New York City, by various loads on pipe resting on sand at considerable depth.

TESTING THE BEARING CAPACITY OF EARTH IN EXCAVATIONS. 1 dr. 1912. (*Engineering Record*, v. 65, p. 584.) Gives methods and results of tests of hardpan.

TESTS OF SETTLEMENTS OF HEAVY LOADS ON SOIL. 1897. (*Engineering Record*, v. 35, p. 332.) Briefly describes Meyer's apparatus for measuring settlements produced by test loads on different small areas of soil.

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TUSKA, GUSTAVE R. Construction of Substructure for Lonesome Valley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad. 1895. (*Transactions*, Am. Soc. C. E., v. 34, pp. 247-252.) Gives maximum pressure per unit area, p. 249.

UNITED STATES—SOILS BUREAU. Bulletins Nos. 1-96. 1895-1913. No more published. *Bulletins Nos. 55* and *76*, contain general information on the soils of the United States. *Bulletin No. 96*, is virtually a complete handbook on the subject.

UNITED STATES—SOILS BUREAU. Field Operations of the Division of Soils, Report, 1st–date; 1899–date. 1900–date. First report is report No. 64 of the Department of Agriculture. Although primarily undertaken to provide the agriculturist with accurate knowledge of the soil, yet it contains much information of value to engineer and builder.

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——Abstract. 1908. (*Minutes of Proceedings*, Inst. C. E., v. 173, p. 357.)

VAN HORN, FRANK R. Landslide Accompanied by Buckling, and Its Relation to Local Anticlinal Folds. 1908. (*Bulletin*, Geol. Soc. of America, v. 20, pp. 625-632.) Describes slide at Cleveland, and discusses causes.

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WADDELL, J. A. L. A Study in the Designing and Construction of Elevated Railroads, with Special Reference to the Northwestern Elevated Railroad and the Union Loop Elevated Railroad of Chicago, Ill. (*Transactions*, Am. Soc. C. E., v. 37, pp. 308-360.) Gives bearing power of Chicago soil, p. 321.

WALMISLEY, A. T. Foundations as Applied to London Buildings and Riverside Foundations. 17 dr. 1898. (*The Builder*, London, v. 74, pp. 514-521.) Shows soil strata underlying London by examinations of borings from localities. Gives much information on safe bearing power of various kinds of ground. Includes also table showing angle at which different materials may safely have sloping banks.

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WHITNEY, MILTON, and BRIGGS, LYMAN J. Electrical Method of Determining the Temperature of Soils. 30 pp. 1897. (*Bulletin No. 7.* U. S. Soils Division.) Wheatstone bridge method.

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WILLMANN, L. VON. Tragfähigkeit eingerammter Pfahle. 1906. (In "Handbuch der Ingenieurwissenschaften," 4 Aufg., 1 Teil, 3 Band, pp. 88-93.)

WILLMANN, L. VON. Tragfähigkeit des Baugrundes. 4 dr., 1 diag. 1906. In "Handbuch der Ingenieurwissenschaften," 4 Aufg., 1 Teil, 3 Band, pp. 12-20.) Discusses standard analytical relations and gives tables of various constants.

WILSON, GEORGE. Some Experiments on Conjugate Pressures in Fine Sand, and Their Variation with the Presence of Water. 4 diag., 2 dr. 1902. (*Minutes of Proceedings*, Inst. C. E., v. 149, pp. 208-222.) Experiments on lateral pressure of earthwork.

WORCESTER, JOSEPH R. Boston Foundations. 6 folding diag., 2 folding dr., 1 folding map. 1903. (*Journal*, Assoc. Eng. Soc., v. 30, pp. 285-302.) Discussion, pp. 302-335. Information is given both in paper and in discussion concerning allowable pressure on soils. Tabulated data are included on kinds of soils and soil combinations to be found in different parts of city.

——Condensed. 1903. (*Engineering News*, v. 49, p. 136.)

WYATT, MATTHEW DIGBY. On the Construction of the Building for the Exhibition of the Works of Industry of All Nations. 1851. (*Minutes of Proceedings*, Inst. C. E., v. 10, pp. 127-191.) Gives pressures allowed, p. 173.

YOUNG, EVELYN HENRY. Foundations in "Black Cotton Soil" in India. 1893. (*Minutes of Proceedings*, Inst. C. E., v. 113, pp. 323-326.) Soil is rich, black earth composed of disintegrated trap, which swells and exerts an enormous pressure, when filled with water.

ZINN, A. S. Truth About the Culebra Cut Slides, Panama Canal. 1913. (*Engineering News*, v. 70, pp. 406-408.) Discusses movement of earth and means of overcoming the difficulties.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM ROBERTS ECKART, M. Am. Soc. C. E.*

DIED DECEMBER 8TH, 1914.

William Roberts Eckart was born on June 17th, 1841, at Chillicothe, Ohio. His father, William R. Eckart, was a merchant of prominence, who, at one time, had large shipping interests on the Lakes and, later, a managing interest in the Putnam Flour Mills, at Zanesville, Ohio. His mother was a member of the Carlisle family, pioneers in the settlement of Ohio.

After attending the public schools of Chillicothe and Cleveland, Mr. Eckart, having determined to take up the Profession of Civil Engineering, took a special course in mathematics at the St. Clair Street Academy, in Cleveland. Accepting the opportunity of serving an apprenticeship in the works of Griffith, Ebert, and Wedge, a firm of good reputation for general mill and steamboat work, he had the good fortune to come under the observation and to obtain the friendly interest of the junior partner, Mr. Wedge, who had had exceptional opportunities, in the shops of Sir Joseph Whitworth, in the manufacture of machine tools requiring extreme accuracy of construction. The effect of Mr. Wedge's insistence that "the best" was to be attained, was not lost on Mr. Eckart, but supplemented his natural ability and became apparent in all his work.

It was natural that his early opportunity to work on steamboat construction and to participate in trial trips, awoke in Mr. Eckart a desire to enter the Government service as a Naval Engineer. The application was made, and in the examinations of June, 1861, he received the highest rating of all applicants of that date.

On July 30th, 1861, he was appointed Third Assistant Engineer and ordered to join the fleet of naval vessels on the Pacific Coast. He arrived in San Francisco in the latter part of 1861, and soon became well known among, and formed friendships with, California's foremost engineers, including Paul Torqua, Joseph Moore, Irving M. Scott, Wallace Hanscom, Huttner, Specht, and many others.

In 1864, after resigning from the Navy on account of ill health, Mr. Eckart entered the employ of H. J. Booth and Company, at San Francisco, under Mr. Irving M. Scott who was Chief Draftsman, and was engaged principally on the design of mining machinery and

* Memoir prepared by C. E. Grunsky, A. M. Hunt, and Hermann F. A. Schussler, Members, Am. Soc. C. E., a Committee of the San Francisco Association of Members of the American Society of Civil Engineers.

steamboat work. As Chief Draftsman with the same firm, Mr. Eckart designed the first locomotive built in California, which was given its trial trip from San Francisco to San José on August 30th, 1865.

In 1869, Mr. Eckart, then holding a license as First-Class Chief Engineer in the Merchant Service, accepted appointment as Draftsman in the Steam Engineering Department of the Mare Island Navy Yard, where, later, he was Foreman Machinist and Superintendent of Steam Machinery. A report by Mr. B. F. Isherwood and Mr. Eckart to the Secretary of the Navy on the relative efficiency of different propellers,* gives an account of the steam machinery, propellers, and dynamometers used in making the necessary experiments.

In 1871, Mr. Eckart left the Navy Yard, and entered into partnership with Prescott, Scheidel and Company, at Marysville, the firm name being afterward changed to Booth and Eckart. This firm specialized in hydraulic, milling, and mining machinery, and by reason of its favorable location did a prosperous business.

In 1876, the firm of Prescott, Scott, and Company, which succeeded that of H. J. Booth and Company, took some large contracts for pumping machinery for the Comstock Mines, and Mr. Eckart was recalled to San Francisco to assist in its design and to superintend its construction. By this time quite a rivalry had sprung up between Prescott, Scott, and Company (The Union Iron Works), and the Risdon Iron Works, in producing designs of machinery to handle the water and ores at Virginia City.

At this date the Sutro Tunnel was "in" 15 500 ft., had 5 000 ft. more to go, and would strike the Comstock Lode just below the 1 600-ft. level. The mine owners were already realizing that nothing but the heaviest and best designed machinery would meet the requirements on all the workings below the tunnel and, therefore, designs for pumps and hoists of large capacity to reach 4 000 ft. below the surface were decided on and called for. Not only the great depth, but also the ventilation and sinking of shafts in virgin ground abounding in large but unknown quantities of hot water to be met and overcome, presented problems unparalleled in engineering experience in any part of the world, and Mr. Eckart had a large share in dealing with and overcoming these conditions.

Having acted as Consulting Engineer for Sutro in sinking four shafts on the line of the tunnel, the investigations then made helped him materially when, in 1876, the orders came for the large pumps and hoists. He spent months in Virginia City experimenting, taking "cards" from pumps where the water would "boil an egg" and the vapors "air-bound the pumps", and where the expansion and contraction strains due to great changes in temperature often wrecked the heaviest castings.

* "On Marine Propulsion," *Transactions*, Inst. of Naval Architects of London, 1872, Vol. XIII, p. 315.

The knowledge thus gained was of great advantage to the firm he represented.

About this time, Mr. Eckart moved to Virginia City and became Consulting Engineer to the "Bonanza Firm", consisting of Messrs. J. W. Mackay, J. C. Flood, J. J. O'Brien, and James G. Fair, which owned or controlled nearly all the "North End" mines. During this time, he was Manager of the Fulton Foundry, at Virginia City. In 1878, he was also appointed United States Deputy Mineral Surveyor for the State of Nevada. During the following two years, a great part of his time was occupied in the underground workings of the Virginia City and Gold Hill Mines, investigating, planning, repairing, and improving the pumps and machinery, as the mines grew deeper and the water kept increasing. While still a resident of Virginia City, he, in connection with Mr. W. I. Salkeld, a noted millwright of the time, designed and built the Bulwer Standard Mill, at Bodie, which, at that time, was one of the largest pan mills for working ore.

During the early part of 1880, Mr. Eckart was appointed a member of the U. S. Geological Survey under Mr. Clarence King, and was given charge of investigating and reporting on "The Mechanical Appliances of the Comstock Lode." On this work, which was practically a labor of love, he spent nearly two years collecting data, testing pumps, engines, and hoists, and making drawings for the Government of all the machinery on the Comstock. The finest instruments procurable in the United States and Europe were used in the various investigations of efficiency. Hydraulic indicators tested and calibrated for from 500 to 5 500 lb. per sq. in. were used in testing the hydraulic pumps designed for the Cholar, Norcross, and Savage Shafts. Chronographs, to measure and record the velocities of engines and pump-rods, accurate to $\frac{1}{500}$ sec., were used on the surface and at a depth of 2 500 ft. The use of this instrument enabled results to be obtained which became of great value; diagrams taken from the heavy rods and pumps on the lower levels gave the clew to the location of the strains which had been so destructive in breaking rods and balance bobs in the past. The velocity curves of the rods and pumps revealed the fact, not theretofore known, and only in recent years properly understood, that the elasticity of long rods could give rise to free vibrations which at times were superposed on the force vibrations and accelerations due to the engines and pumps, so that maximum vibrations and strains resulted at certain parts of the rods, which exceeded the elastic limit of the timber of which the rods were constructed. Changes were thus determined in the location of balance bobs and weights, which increased the efficiency of the pumps and checked the destruction of the rods and bobs. Some of these velocity diagrams, as well as illustrations of the largest pump engines and the hydraulic pumps, were

published by Professor A. Riedler, of Berlin, while acting as Commissioner of Mines to the German Government in 1893.

In 1881, Mr. Eckart removed to San Francisco and opened offices there as a Consulting and Constructing Engineer and, during the following eight or ten years, some of the largest and most important mining plants were designed and constructed under his supervision. The pumping engine for the Ontario Mine, with perhaps the largest Cornish pump for deep mining ever built in the United States, was constructed from his designs during this period. The pumps were 20 in. in diameter, with 10-ft. stroke; two pumps at each station operated from one pump rod, 2 000 ft. long. In 1881, he began, for Haggin and Tevis, plans for all the Anaconda Copper Works, Hoists, and Reduction Works, and during the following seven years he designed all the mining work and mills of this firm. The Anaconda Reduction Works, in Montana, which were started as a silver mill with a capacity of less than 350 tons per day, were increased in size and changed by additions until, in 1888, they were capable of working 3 000 tons per day. He designed and carried out much other work for the same firm.

In 1883, the Union Iron Works was changed to an incorporated company, and Mr. Eckart was retained as Consulting Engineer in matters pertaining to the propelling power of the Government vessels built by that Company. He was present at and assisted in conducting nearly all the preliminary and Government trials of these vessels.

In 1899, Mr. Eckart was appointed Consulting Engineer to the Standard Electric Company, and afterward became Engineer in charge of construction as well. He continued to act as Consulting Engineer after the completion of the plant, and after the Pacific Gas and Electric Company acquired his property as a part of its great power system, he was retained in the same capacity by the latter company, in connection with the hydro-electric branch of the work.

In 1907, the Snow Mountain Water and Power Company engaged Mr. Eckart as Consulting and Constructing Engineer, and he was connected with that company until he retired from active business at the close of 1913. He continued in the employ of the Pacific Gas and Electric Company, however, to the day of his death, which occurred on December 8th, 1914, at the residence of his son, Mr. W. R. Eckart, Jr., in Palo Alto, Cal. He is survived by his widow, Harriet Louise Eckart, and four children, W. R. Eckart, Jr., of the Leland Stanford, Jr., University, Charles F. Eckart, of Olaa, Hawaii, Nelson A. Eckart, M. Am. Soc. C. E., of San Francisco, and Mrs. C. E. Hume, of Piedmont, Cal.

Mr. Eckart had unusual talent as an experimenter and investigator. His methods were thorough, painstaking, and accurate, and

his records of tests and experiments could always be regarded as absolutely trustworthy. Most of the special instruments and devices needed in his various investigations were designed and made by himself. Among such instruments was a "Chronograph for Engineering Purposes, with the Hipp Escapement", described under that title by him in a paper* presented before the American Society of Mechanical Engineers, in 1882. In the discussion of that paper, the late Robert H. Thurston, M. Am. Soc. C. E., of Cornell University, speaks of Mr. Eckart as follows: "He is a very careful worker. In fact, I have not met in the profession a man who seemed better adapted to the application of fine measurements to experimentation on the steam-engine." This instrument was devised for use in the investigations made by Mr. Eckart on the Cornish pumps of the Comstock Lode, and was used subsequently in other investigations requiring accurate time measurements.

As a man and among his colleagues, Mr. Eckart had the admiration of all who knew him. He was a hard worker, always going to the bottom of his problems; and the mass of data which he had accumulated and the results of his ripe experience, were at the service of his colleagues whenever he was appealed to. His friendly help has been appreciated and will be remembered by all who have worked or conferred with him.

Mr. Eckart was honored by membership in the following Engineering Societies: American Society of Mechanical Engineers, April, 1882; Institution of Mechanical Engineers, London, January, 1878; Society of Naval Architects and Marine Engineers, May, 1893; American Society of Naval Engineers; and Associate Member of the Institute of Naval Architects, London.

Mr. Eckart was elected a Member of the American Society of Civil Engineers, on January 5th, 1881.

* *Transactions*, Am. Soc. Mech. Engrs., Vol. 3, p. 184.

PAPERS IN THIS NUMBER

"SUGGESTED CHANGES AND EXTENSION OF THE UNITED STATES WEATHER BUREAU SERVICE IN CALIFORNIA." GEORGE S. BINCKLEY and CHARLES H LEE.

"CINDER CONCRETE FLOOR CONSTRUCTION BETWEEN STEEL BEAMS." HAROLD PERRINE and GEORGE E. STREHAN. (To be presented Mar. 3d. 1915.)

PROGRESS REPORT OF THE SPECIAL COMMITTEE TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS.

PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

"Huacal Dam, Sonora, Mexico." H. HAWGOOD	Apr.,	1914
Discussion. (Author's Closure.)	Aug., 1914, Feb.,	1915
"Some Principles Relating to the Administration of Streams." CLARENCE T. JOHNSTON	May,	1914
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"External Corrosion of Cast-Iron Pipe." MARSHALL R. PUGH	Aug.,	1914
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"The Clarification of Sewage by Fine Screens." KENNETH ALLEN	Aug.,	"
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"Water Supply of the San Francisco-Oakland Metropolitan District." H. T. CORY	Sept.,	1914
Discussion	Nov., Dec., 1914, Jan., Feb.,	1915
"The Lock 12 Development of the Alabama Power Company, Coosa River, Alabama." E. L. SAYERS and A. C. POLK	Sept.,	1914
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"Investigation of the Performance of a Reaction Turbine." R. L. DAUGHERTY	Oct.,	1914
Discussion	Jan., Feb.,	1915
"The Differential Surge Tank." RAYMOND D. JOHNSON	Oct.	1914
Discussion	Oct.,	"
"Penstock and Surge-Tank Problems." MINTON M. WARREN	Oct.,	"
Discussion	Jan., Feb.,	1915
"Submerged Pipe Work at Portland, Oregon." D. D. CLARKE	Nov.,	1914
Discussion	Jan., Feb.,	1915
"History of Little Rock Junction Railway Bridge of the St. Louis, Iron Mountain and Southern Railway Company, Over the Arkansas River, at Little Rock, Arkansas, 1883-1914." C. E. SMITH	Nov.,	1914
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Discussion	Jan., Feb.,	1915
"The Water-Proofing of Solid Steel-Floor Railroad Bridges." SAMUEL TOBIAS WAGNER	Dec.,	1914
Discussion	Jan., Feb.,	1915
"Nomographic Solutions for Formulas of Various Types." R. C. STRACHAN	Dec.,	1914
Discussion	Feb.,	1915
Progress Report of the Special Committee on Materials for Road Construction and on Standards for Their Test and Use	Dec.,	1914
Progress Report of the Special Committee on Steel Columns and Struts	Dec.,	"
Discussion	Feb.,	1915
Progress Report of the Special Committee to Investigate the Conditions of Employment of, and Compensation of, Civil Engineers	Dec.,	1914
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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

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(INSTITUTED 1852)

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MARCH, 1915

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NEW YORK 1915

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TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner.

ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

ON FLOODS AND FLOOD PREVENTION: C. McD. Townsend, John A. Bensel, T. G. Dabney, C. E. Grunsky, Morris Knowles, J. B. Lippincott, Daniel W. Mead, John A. Ockerson, Arthur T. Safford, Charles Saville, F. L. Sellew.

TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Robert W. Hunt, George W. Kittredge, Paul M. LaBach, C. G. E. Larsson, William McNab, G. J. Ray, Albert F. Reichmann, F. E. Turneaure, J. E. Willoughby.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

February 17th, 1915.—The meeting was called to order at 8.30 P. M.; Austin L. Bowman, M. Am. Soc. C. E., in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 158 members and 22 guests.

A paper by William G. Grove, Esq., and Henry Taylor, Assoc. M. Am. Soc. C. E., entitled "Reconstruction of the Norfolk and Western Railway Company's Bridge over the Ohio River at Kenova, West Virginia," was presented by Mr. Grove and illustrated with lantern slides. The paper was discussed by Messrs. T. Kennard Thomson, F. W. Skinner, Otis E. Hovey, and W. J. Boucher. A communication on the subject from C. H. Cartlidge, M. Am. Soc. C. E., was read by the Assistant Secretary.

Adjourned.

March 3d, 1915.—The meeting was called to order at 8.30 P. M.; Vice-President Gardner S. Williams in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 132 members and 21 guests.

The minutes of the Annual Meeting, January 20th, and of the meeting of February 3d, 1915, were approved as printed in *Proceedings* for February, 1915.

The Chairman appointed Messrs. A. H. Van Cleve, F. W. Skinner, and J. H. Granbery as Tellers to canvass the ballot on the following proposed amendment to the Constitution:

Amend Article VII—Nomination and Election of Officers—as follows:

Strike out Section 1, and substitute the following:

"The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year."

Strike out the first paragraph of Section 2, and substitute the following:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, shall be appointed by the meeting to serve for two years. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve Districts; and these, with the last five living Past-Presidents of the Society, shall be a committee to nominate officers for the Society."

Strike out the word "and" in the fifteenth line of the third paragraph of Section 2, and after the figure "7" add: "8, 9, 10, 11, 12, and 13."

The Tellers reported as follows:

Total number of ballots received		1 178
Unsigned	13	
From those in arrears for dues	8	
Stamped, not signed	2	23
Ballots canvassed		1 155
Voting Yes		1 066
No		83
Blank ballots (not marked)		6

The Chair declared the amendment carried.

A paper, entitled "Cinder Concrete Floor Construction Between Steel Beams", by Harold Perrine and George E. Strehan, Juniors, Am. Soc. C. E., was presented by Mr. Strehan and illustrated with lantern slides. A communication on the subject from K. M. Boorman, Esq., was read by the Assistant Secretary, and the paper was discussed orally by Messrs. Guy B. Waite, T. Hugh Boorman, Albert Oliver, Ira H. Woolson, M. S. Falk, and George E. Strehan.

The Assistant Secretary announced the election of the following candidates on March 2d, 1915:

As Members

Alfred Francis Théard, New Orleans, La.
John Wilson, Austin, Tex.

As Associate Members

Gault Applegarth, Baltimore, Md.
Ernest Ruby Bear, Wheeling, W. Va.
Arthur Bryan, Birmingham, England
Frank Joseph Conlon, Brooklyn, N. Y.
William Carey Cram, Jr., Raleigh, N. C.
Robert Menees Davis, Newman, Cal.
Carl S. Donaldson, Beaver Falls, Pa.
James Lumnsden Ferebee, Atlantic City, N. J.
Douglass Hewitt Ferry, San Diego, Cal.
Roy Warner Gausmann, Brown Station, N. Y.
Herbert Ellsworth Howes, Minneapolis, Minn.
Charles Reginald Hughes, Lima, Ohio
Herbert Alfred Jackson, Melbourne, Victoria, Australia
Wesley Eugene King, St. Paul, Minn.
Percy Hugh Marshall Macintosh, Sydney, New South Wales, Australia
John Alexander Norris, Wharton, Tex.
Benjamin Simpson Philbrick, Santa Rosalia, Chih., Mexico
Nathan Schein, Pittsburgh, Pa.
Frederick Robert Sites, Shanghai, China
James Elmo Smith, Urbana, Ill.
Julien Smith, Selma, Ala.
Thomas Henry Strate, Aberdeen, S. Dak.
Harold Warner Streeter, Cincinnati, Ohio
Carl Stuetzel, Jr., Boston, Mass.
George Eliott Sturt, Sault Ste. Marie, Mich.
Oscar Gowen Thurlow, Birmingham, Ala.
Arthur John Turner, Long Lake, Wash.
Guy Vroman, Larchmont, N. Y.

As Juniors

Frank Stork Altman, Atchison, Kans.
Dario Bressane, Campanha, Minas Geraes, Brazil
Carlyle Hugo Bryson, Lima, Ohio
Ivan Charles Crawford, Boulder, Colo.
John James Gault, Bocas del Toro, Panama
Lewis Merrick Hammond, Provo, Utah
William John Krefeld, New York City
Joseph Mastella LeGrand, Clarkdale, Ariz.
Samuel Brooks Morris, Pasadena, Cal.
Russell Moore Robinson, Dayton, Ohio
Harold August Hastrup Schultz, Detroit, Mich.
Charles Maria Weber, Ithaca, N. Y.
Alphonse Mueller Westenhoff, Cincinnati, Ohio
Henry Willcox, West New Brighton, N. Y.

The Assistant Secretary announced the transfer of the following candidates on March 2d, 1915:

From Associate Member to Member

Harry Edmund Bilger, Springfield, Ill.
Con Morrison Buck, Topeka, Kans.
Adam Leonard Bush, Los Angeles, Cal.
Ross Miller Coomer, Sioux City, Iowa
Louis Christian Datz, New Orleans, La.
Harry Westbrook DeGraff, Amsterdam, N. Y.
Thomas Alvin Gilkey, New Castle, Pa.
Carleton Greene, New York City
Oscar Llewellyn Grover, Washington, D. C.
Julius Lilien Jacobs, Houston, Tex.
Charles Augustin Woodley Musson, Butte, Mont.
Ralph Fenno Proctor, Baltimore, Md.
Frederic Adams Reimer, East Orange, N. J.
Thomas Riggs, Jr., Washington, D. C.
Elwyn Eggleston Seelye, New York City
Adelbert Alonzo Weiland, Pueblo, Colo.
Thad Loren Wilson, New York City

From Junior to Associate Member

James Madison Barker, Boston, Mass.
John Cleaveland Beebe, Richfield, Idaho
John Henry Bringhurst, Ann Arbor, Mich.
Massena Laron Culley, Jackson, Miss.
Ralph Samuel Gram, Toledo, Ohio

George Elias Halstead, LaFayette, Ind.
Horace Parlin Hamlin, New York City
Charles Jacob Hyer, Tampa, Fla.
Frank Henry Macy, Dansville, N. Y.
Wilfred Lincoln Rowe, Meadow Creek, Wash.
William Lee Selmer, New York City
Horatio Seymour, Jr., Bronxville, N. Y.

From Junior to Associate

Roger Leroy Morrison, College Station, Tex.

The Assistant Secretary announced the following deaths:

Frank Olin Marvin, of Lawrence, Kans., elected Member, May 5th, 1897; died February 6th, 1915.

Saint George Henry Cooke, of Philadelphia, Pa., elected Associate Member, January 5th, 1909; died January 12th, 1915.

Harold Hansen Fitting, of San Francisco, Cal., elected Junior, May 2d, 1911; Associate Member, September 3d, 1913; died January 7th, 1915.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

March 2d, 1915.—The Board met at 4.40 P. M., President Marx in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Bontecou, Coleman, Crocker, Davies, Endicott, Greiner, Herschel, Montfort, Ockerson, Swain, Tuttle, and Williams.

Mr. Hunt presented a draft of a Bill prepared by a Joint Committee of the National Engineering Societies with the resolutions adopted by that Committee February 4th, 1915, as follows:

"*Resolved:* That the Members of this Joint Committee present to their respective governing bodies an official copy of the Act as finally adopted by the Committee, with the suggestion that it be used, should it become necessary, in connection with legislation on the subject.

"*Resolved:* That the Joint Committee having completed the task assigned to it, in so far as it has been able to do so, requests that it be discharged."

Robert Ridgway and Chas. Warren Hunt were, in accordance with this resolution and their request, discharged as representatives of this Society on said Committee.

Edwin Duryea, Jr., was appointed a member of the Committee of Management of the International Engineering Congress, 1915, to fill the vacancy caused by the election of Mr. Marx, by virtue of which he becomes a member of the Committee on Participation.

A proposed amendment to the Constitution of the Philadelphia Association of Members of the Society was approved.

A proposed revision of the Constitution of the Atlanta Association of Members of the Society was approved.

General Grenville Mellen Dodge was elected an Honorary Member by the unanimous vote of the Board of Direction and of all living Past-Presidents.

The resignations of 2 Members, 4 Associate Members, and 3 Juniors were accepted.

Ballots for membership were canvassed, resulting in the election of 2 Members, 28 Associate Members, and 14 Juniors, and the transfer of 1 Junior to the grade of Associate, and 12 Juniors to the grade of Associate Member.

Seventeen Associate Members were transferred to the grade of Member.

Applications for membership were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

April 7th, 1915.—8.30 P. M.—A regular business meeting will be held, and a paper by R. M. Strohl, Jun. Am. Soc. C. E., entitled "The St. John Levee and Drainage District of Missouri", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

April 21st, 1915.—8.30 P. M.—At this meeting a paper by Adolph F. Meyer, M. Am. Soc. C. E., entitled "Computing Run-Off from Rainfall and Other Physical Data", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Annual Convention of the Society will be held in San Francisco, Cal., September 16th, 17th, and 18th, 1915, being the Thursday, Friday, and Saturday immediately preceding the International Engineering Congress, which is to be held during the week beginning September 20th.

Three of the other National Engineering Societies, under whose auspices the International Engineering Congress is to be held, will also hold meetings in San Francisco at about that time.

Arrangements will be made for a special train, and possibly more than one train, to accommodate the members of all of these Societies who wish to attend their own meeting as well as the Congress.

This matter is in the hands of a Joint Committee of representatives of these organizations, and it is expected that within a short time a circular will be mailed to the entire membership of these Societies (some 28 000), giving in detail a schedule of train service, railroad and hotel rates, etc.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the

membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Meeting of Presidents of Local Associations

A meeting of the Presidents or representatives of the fourteen Local Associations of Members of the American Society of Civil Engineers was held at the House of the Society on January 19th, 1915, the day preceding the Annual Meeting. A Report in Full of the Proceedings of this meeting will be published in the April *Proceedings*.

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meeting)

December 18th, 1914.—The Tenth Annual Meeting was called to order; President Snyder in the chair; E. T. Thurston, Jr., Secretary; and present, also, 80 members and guests.

Mr. C. E. Grunsky, for the Committee on Headquarters for the Annual Convention, reported that satisfactory arrangements had been made with the Hotel St. Francis. On his motion, it was voted to offer the assistance of the Association to the Board of Direction in the selection of the Entertainment Committee for the Convention.

Mr. Thurston, for his committee, reported that the Engineers' Club of San Francisco would be pleased to make arrangements with the Association relative to holding the weekly luncheons in its new quarters as soon as they were ready.

The Reports of the Treasurer and Secretary were read and adopted.

On motion, duly seconded, President Snyder was instructed to appoint a Committee of three to draft suitable resolutions of sympathy to the family and relatives of the late W. R. Eckart, M. Am. Soc. C. E. Messrs. C. E. Grunsky, Hermann Schussler, and A. M. Hunt were subsequently appointed.

The following officers were elected: President, A. M. Hunt; Vice-President, M. C. Couchot; Secretary, E. T. Thurston, Jr.; and Treasurer, P. E. Harroun.

Mr. J. D. Galloway addressed the meeting on "The Installation of Unit No. 5 at the Las Plumas Power Station of the Great Western Power Company", illustrating his remarks with stereopticon views.

The paper was discussed briefly by Messrs. A. M. Hunt, Rhodin, Homberger, Doble, and Galloway.

Adjourned.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

February 6th, 1915.—The meeting was called to order; President Vincent in the chair; Roger W. Toll, Secretary; and present, also, 28 members and guests.

The minutes of the January, 1915, meeting were read and approved.

The Secretary reported that the ballot taken regarding the day of the monthly meeting and time of the dinner, showed a decided preference for Saturday evening and a 6 o'clock dinner.

Communications were read from the Secretary of the Society, Mr. G. N. Houston, and others.

President Vincent reported on the Conference of the Presidents of the Local Associations held in New York City on January 19th, 1915.

A report for the Committee on Licensing Civil Engineers was presented by Mr. John E. Field, and the subject was discussed by Messrs. Leonard Metcalf, H. S. Crocker, and others.

A paper on "The Drainage of Irrigated Lands" was presented by Mr. D. W. Murphy, who illustrated his remarks with stereopticon views, and there was a general discussion of the subject.

A vote of thanks was tendered Mr. Murphy for his interesting paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. P. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution

adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

At its meeting of January 6th, 1915, the Board of Direction considered and approved the proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

The Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.) has been organized with the following officers: Frank M. Kerr, President; J. F. Coleman, and W. B. Gregory, Vice-Presidents; A. M. N. Blamphin, Treasurer; and L. C. Datz, Secretary.

Northwestern Association

At its meeting of November 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.). F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

At the meeting of the Association on October 5th, 1914, the following officers were elected for the ensuing year: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow and William Hunter; Directors, John Sterling Deans, John W. Ledoux, Edgar Marburg, and H. S. Smith; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

Portland, Ore., Association

At the meeting of the Association on October 21st, 1914, the following officers were elected for the ensuing year: President, George C. Mason; First Vice-President, W. S. Turner; Second Vice-President, John T. Whistler; Treasurer, G. B. Hegardt; and Secretary, Charles J. McGonigle.

St. Louis Association

At its meeting of October 7th, 1914, the Board of Direction considered and approved the proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensuing year: President, R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 515 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers. Ulysses B. Hough is President.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Stresses in Railroad Track

January 19th, 1915.—The meeting was held at the House of the Society. Present, A. N. Talbot (Chairman), William McNab, G. J. Ray, and J. E. Willoughby (of the Committee) and also W. M. Dawley

and P. H. Dudley (of the Committee of the American Railway Engineering Association).

The Chairman presented informally the results of the tests already made on the Illinois Central Railroad to determine the equilibrium depression curve of rail, tie, and roadbed, and strain in rail, and made an informal oral report of the results of the test work and of the instruments used and devised. The results of the tests were discussed by members of the Committee. Dr. Dudley presented, in printed form, the results of tests, on the New York Central Railroad, to find the stresses in rail with locomotive running at various speeds. A Report of Progress, for presentation to the Annual Meeting, was discussed and adopted. The Chairman was authorized to present a budget to the Board of Direction as a basis for an appropriation for the coming year.

Special Committee on Floods and Flood Prevention

January 19th, 1915.—The third meeting of the Committee was held at the House of the Society, at 2 P. M. Present, C. McD. Townsend (Chairman), John A. Bensel, and John A. Ockerson. The draft of a report was read and discussed, and it was finally decided that, with such a limited number of members of the Committee present, it was advisable merely to report progress at the Annual Meeting of the Society.

Special Committee on Materials for Road Construction

January 21st, 1915.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, Charles J. Tilden, and A. H. Blanchard (Secretary).

A communication from the Secretary of the Society, dated December 7th, 1914, in reference to the granting by the Board of Direction of the appropriation of $48.38 for laboratory research work, was read.

The Regulations in reference to Special Committees, adopted by the Board of Direction on January 6th, 1915, were discussed.

The Chairman outlined the work of 1915 in reference to non-bituminous road materials, and announced the continuance of the same sub-committees as appointed in 1914.

The Chairman requested that lists of foreign non-members of the Society, to whom the 1915 Report should be forwarded with a view to soliciting discussion, should be sent to the Secretary on or before February 1st, 1915.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 312 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From February 2d to March 1st, 1915)

DONATIONS*

PRACTICAL IRRIGATION AND PUMPING:

Water Requirements, Methods of Irrigation, and Analyses of Cost and Profit. By Burton P. Fleming, Assoc. M. Am. Soc. C. E. Cloth, 8¼ x 5½ in., illus., 16 + 226 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $2.00.

The solution of the irrigation problem of the vast uncultivated areas in the West, lying on the higher benches or mesas adjacent to irrigated valleys and above high-line canals, lies, the author states, in the pumping of surface or subsurface water supplies. In this volume, therefore, he presents the subject of irrigation chiefly from the pumping standpoint, discussing in detail the design, operation, installation, and cost of a small pumping plant for such purpose, which he hopes will be helpful to the contractor who specializes in pumping machinery and the engineer who is called on to design the central station plant. The author, it is stated, has had more than eight years' experience in irrigation work covering most of the Western States, which experience he has embodied in this volume. The Chapter headings are: The Amount of Water Required; Sources of Supply; The Flow of Underground Water; Strainers; Well Sinking; Pumps, Pumping Machinery, and Appliances; Centrifugal Pumps; Different Types of Installation for Centrifugal Pumps; Typical Plants Not Using Centrifugal Pump; Cost of Pumping; The Question of Cost and Profit on a Small Farm Irrigated by Pumped Water; Reservoirs; Prime Movers; The Central Station Pumping Plant; Windmills; Appendix: Partial List of Manufacturers and Dealers in Machinery and Appliances Used in Irrigation Pumping; Index.

AIR, WATER, AND FOOD

From a Sanitary Standpoint. By Alpheus G. Woodman and John F. Norton. Fourth Edition, Revised and Rewritten. Cloth, 9¼ x 6 in., illus., 5 + 248 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1914. $2.00.

The subject-matter of this book deals chiefly, it is stated, with the chemical analysis of air, water, and food, in their relations to the needs of daily existence, and to such portions of the chemistry of sanitation as come directly under individual control or which require the education of individuals in order to carry out sanitary measures. The preface states that since the last edition of the work was published, in 1909, distinct advances have been made in analytical methods and in the interpretation of results, all of which has necessitated, in this edition, the complete rewriting of the chapters on air and water as well as the section on milk, a revision of the older methods discussed, and numerous additions, in order to bring the contents up to the latest practice. A bibliography of the subjects discussed is included. The Chapter headings are: Three Essentials of Human Existence; Air and Health; Air: Analytical Methods; Water: Its Relation to Health, Its Source and Properties; Safe Water and the Interpretation of Analyses; Water: Analytical Methods; Food in Relation to Human Life, Definition, Sources, Classes, Dietaries; Adulteration and Sophistication of Food Materials; Analytical Methods; Appendix A, Tables; Appendix B, Reagents; Bibliography; Index.

DESIGNING CHART FOR STEEL OR WOOD BEAMS.

By Charles W. Schubert, Assoc. M. Am. Soc. C. E. Printed on Veneered Celluloid, 12 x 7¾ in. Cleveland, Ohio, The Author, 1914. $5.00.

This chart, the designer states, may be considered as a designer's handbook on one sheet, as it is intended to save time in handling problems of design ordinarily solved by reference to structural handbooks. The fundamental principle involved in solving formulas by use of this chart is to align two assumed variables and read the third on its proper axis. It is said to cover all the standard and Bethlehem sections of I-beams, channels, and angles, as well as the new Carnegie "supplementary" sections, and wood beams may also be designed by its use. The graduations on the chart show the allowed uniform load and load per linear foot for any

* Unless otherwise specified, books in this list have been donated by the publishers.

span, spacing for any load per square foot of floor, maximum bending moments, section moduli for steel shapes, and moment of resistance of wood beams for any fiber stress.

A MODEL FIRE-PROOF FARM HOUSE OR COUNTRY HOME:

Practical Suggestions for Economical and Enduring Construction with Complete Plans and Specifications of a Model Building. By A. L. A. Himmelwright, M. Am. Soc. C. E. Paper, 9¼ x 11¾ in., illus., 91 pp. New York, The Neale Publishing Company, 1913. $1.00. (Donated by the Author.)

The object of this volume, as stated by the author, is to present full, practical, and correct information on every phase of building an inexpensive fireproof country home without the aid of an architect. The subject-matter is said to be wholly original and based on the author's long experience in the construction of buildings. It includes general and detailed plans and complete specifications, a practical treatment of such important subjects as location, drainage, local water supply, pipe sizes, etc., together with quantities, complete and accurate costs, and other data based on the actual costs of a model building constructed in 1909, of which careful record was kept. The Contents are: Location, Water Supply, Drainage, Excavating and Grading; The Plans; Practical Suggestions; A Model Sixteen-Room Building; Specifications; Schedule of Material; A Model Ten-Room Building; Appendix: A Partial List of Names and Addresses of Reliable Concerns Equipped to Furnish Materials for These Model Buildings.

UNIVERSAL SAFETY STANDARDS, MACHINE SHOP AND FOUNDRY:

A Reference Book of Rules, Drawings, Tables, Formulas, Data, and Suggestions for Use of Architects, Engineers, Superintendents, Foremen, Inspectors, Mechanics and Students. By Carl M. Hansen. Second Edition, Revised and Enlarged. Leather, 7¾ x 5¼ in., illus., 312 pp. New York, Philadelphia, London, Universal Safety Standards Publishing Company, 1914. $3.00.

In the first edition of this volume, published in November, 1913, the question of safety standards was treated, it is stated, in a general manner. In the present volume, which is the first of a series of books on the subject to be issued under the direction of the Workmen's Compensation Service Bureau, the subject is treated from the standpoint of safety in machine shops and foundries. The author's aim has been to present, for ready reference, and in detail from the construction of the plant through the shipping room, conditions ordinarily found in machine shops and foundries, with the proper safeguards applied, and to illustrate each description with drawings which show clearly the application of the principles of such safeguarding. Attention is especially called by the author to the treatment of stairways, steam engine guarding, and grinding wheel protection. The Contents are: Part I, General Safety Standard; Part II, Machine Shop; Part III, Foundry; Part IV, Rules for Practice: Machine Shops, Metal Shops and Foundry; Index.

GRAPHIC METHODS FOR PRESENTING FACTS.

By Willard C. Brinton. (Works Management Library.) Cloth, 10¼ x 7¼ in., illus., 12 + 371 pp. New York, The Engineering Magazine Company, 1914. $4.00.

The author states that the proper kind of a chart is infinitely better than a column of figures in making comparisons of various items of business, and, therefore, an effort has been made to produce, in this volume, a work which will serve as a handbook for any one who may have to prepare charts for reports, for magazine illustration, or for advertising. The subject, it is stated, is presented from the point of view of the business man, technical terms and mathematics being omitted as much as possible, but it is hoped that the book may prove useful to the engineer, the biologist, and the statistician. Although the text contains much detailed information concerning the methods of charting, the contents have been arranged, the author states, so that the busy reader may get the gist of the contents from the illustrations and the titles and sub-titles. The Contents are: Component Parts; Simple Comparisons; Simple Comparisons Involving Time; Time Charts; Curve Plotting; Comparison of Curves; Component Parts Shown by Curves; Cumulative or Mass Curves; Frequency Curves, Correlation; Map Presentations; Maps and Pins; Curves for the Executive; Records for the Executive; Corporation Financial Reports: General Methods; A Few Cautions; Index.

ELEMENTS OF HYDRAULICS.

By S. E. Slocum. Cloth, 9¼ x 6¼ in., illus., 11 + 294 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $2.50.

This book, it is stated, is intended as a modern presentation of the fundamental principles of hydraulics with application to such recent important works as the Catskill Aqueduct, the New York State Barge Canal, and the power plants at Niagara Falls and Keokuk. The design of turbines is not included, it is said, but the recent works of Zowski and Baashuus are presented in such a manner that the young engineer is enabled to make an intelligent choice of the type of development and selection of runner. At the end of each section, the author has included a collection of typical modern problems in hydraulics, and he has also compiled and tabulated many useful hydraulic data which are given at the end of the book. The Contents are: Section I, Hydrostatics; Section II, Hydrokinetics; Section III, Hydrodynamics; Section IV, Hydraulic Data and Tables; Index.

PROCEEDINGS OF THE SIXTH NATIONAL CONFERENCE ON CITY PLANNING.

Toronto, May 25-27, 1914. Cloth, 9½ x 6½ in., illus., 361 pp. Cambridge, Mass., The University Press, 1914. $2.00. (Donated by the National Conference on City Planning.)

This volume contains the Proceedings of the first meeting of the National Conference on City Planning held in Canada. At this meeting, which was officially recognized by the Canadian Government, the various Provinces, and the City of Toronto, the work of city planning and of municipal administration was discussed by officials and engineers, making possible the exchange of experiences in the handling of various municipal problems such as water fronts, city financing, housing, rapid transit, etc., all of which have been compiled and are presented in this book. An exhibition of Canadian town planning was also shown by the Commission of Conservation. The Contents are: Remarks at the Opening of the Conference; Basic Principles of Water Front Development, by Robert Gourlay; Certain Aspects of City Financing and City Planning, by Andrew Wright Crawford; Progress of the Year in City Planning, by Flavel Shurtleff; Protecting Residential Districts, by Lawrence Veiller; A Town Planning Act for Canada, by J. H. Burland; Canada and the United States as a Field for the Garden City Movement, by G. Trafford Hewitt; Provision for Future Rapid Transit, by J. V. Davies; Utility of the Motor Bus and Municipal Problems Pertaining to Its Operation, by J. A. McCollum; The New York Rapid Transit Problem, by George McAneny; Size and Distribution of Playgrounds and Similar Recreation Facilities in American Cities, by Henry V. Hubbard; Remarks at the Closing Dinner; Conference Business; Appendix: Extracts from City Charter of Halifax, Nova Scotia.

STRUCTURAL STEEL DRAFTING AND ELEMENTARY DESIGN.

By Charles D. Conklin, Jr., Assoc. M. Am. Soc. C. E. Cloth, 9¼ x 12 in., 7 + 154 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915.

This book, the preface states, deals with the preparation of shop detail drawings of structural steelwork, and is presented for use as a guide in the technical school drafting-room. No claim is made that all information relating to shop detail drawings is given, but sufficient examples and instructions are included to familiarize the beginner with drawing-room practice of the best and largest structural steel firms and to give him a knowledge of the methods used in detailing. The author has worked out a number of typical simple designs and has had shop drawings for the same prepared, thereby presenting to the student the entire work of the structural steel drafting-room, including the engineering department. The Contents are: Drafting-Room; Estimating and Designing; Equipment, Lettering, Assembling Marks; Ordering Material: Detailing Elementary Shapes; Rivets and Riveted Connections; Beam and Column Detailing; Wall Girder and Columns—Design and Detail; Design and Detail of Steel Roof; Design and Detail of Deck Plate Girder Railway Bridge; Shop Detail of Through Girder Bridge; Design and Detail of Country Highway Bridge; Design of Through Riveted Railway Span; Detail Drawings for Through Railway Bridges; Hip and Valley Details; Appendix: General Specifications for Steel Railway Bridges; Index.

CITY OFFICIALS OF THE UNITED STATES

Comprising all Cities and Towns Having More Than 8 000 Inhabitants, Revised February 1st, 1915. Paper, 6 x 3¾ in., 39 pp. New York, Engineering News, 1915.

The subject-matter of this little volume is arranged alphabetically by cities and includes the name of the Mayor or Chief Executive of each town, the names

of the City Clerk, City Engineer, and of the Superintendent or official in charge of the water supply. The form of government of each town is also given, whether commission government, city manager plan, etc., as well as the population according to the United States Census of 1910.

STRUCTURAL ENGINEERING.

By J. E. Kirkham, Assoc. M. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., illus., 4 + 669 pp. Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1914. $5.00.

The preface states that this book is intended as a textbook for college students and as a manual of structural engineering for practical men. The subject-matter treats of simple structures only, the author, it is stated, having in preparation a second volume which will treat of "Higher Structures", and relates to the determination of stresses, the selection of material, the drawing of details, the design of foundations, concrete floors, roofs, etc. The designs included in the text are stated to be entirely the work of the author and made especially for this book, the usual methods of designing being presented so that they may be read and understood without the use of a dictionary, glossary, or compendium of theoretical mechanics. The author, it is stated, has arranged the drawing-room exercises throughout the text in order that the work in the classroom and workroom will go hand in hand. The Contents are: Preliminary; Structural Draughting; Fundamental Elements of Structural Mechanics; Theoretical Treatment of Beams; Theoretical Treatment of Columns; Rivets, Pins, Rollers and Shafting; Maximum Reactions, Shears, and Bending Moments on Beams and Trusses and Stresses in Trusses; Graphic Statics; Influence Lines; Design of I-Beams and Plate Girders; Design of Simple Railroad Bridges; Design of Simple Highway Bridges; Skew Bridges, Bridges on Curve, Economic Height and Length of Trusses and Stresses in Portals; Design of Buildings; Index.

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Coal Gas Residuals. By Frederick H. Wagner. New York and London, 1914.

Keister's Corporation Accounting and Auditing: A Practical Treatise on Higher Accounting. By D. A. Keister. With an Introduction by Henry C. White. Fourteenth Edition. Cleveland, 1912.

Handbook of Tables and Formulas for Engineers. Compiled by Clarence A. Peirce. With Mathematical Sections by Walter B. Carver. New York and London, 1914.

The American Year Book: A Record of Events and Progress 1914. Edited by Francis G. Wickware, with Co-operation of a Supervisory Board Representing National Learned Societies. New York and London, 1915.

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HIRAI, KIKUMATSU. Engr., Imperial Govt. Rys., Tomoge Hiratsukamura Ebaragori, Tokyo, Japan.

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HUNTSMAN, FRANK C. Div. Engr., C., B. & Q. R. R., Box 1397, Alliance, Nebr.

KELLY, JOHN ARTHUR. 900 South 10th St., Philadelphia, Pa.

MCCLURE, HUNTER. Care, Interstate Commerce Comm., 731 Wells Fargo Bldg., San Francisco, Cal.

MCENTIRE, LLOYD. Div. Highway Engr., New Jersey State Dept. of Public Roads, 244 North Warren St., Trenton, N. J.

MARKS, EDWIN HALL. First Lieut., Corps of Engrs., U. S. A., Manila, Philippine Islands.

MILLS, GUY G. Instr. in Structural Eng., Univ. of Illinois, Urbana, Ill.

MUNKELT, FREDERICK HERMANN. Care, Petroleum Iron Works Co., Sharon, Pa.

NAJJAR, SIMON ABRAHAM. Eng. Insp., Board of Water Supply, New York City, 438 DeGraw St., Brooklyn, N. Y.

SEGURA, VALERIANO. Dist. Engr., Bureau of Public Works, Province of Antique, San José de Buena Vista, Philippine Islands.

STARR, WILLIAM H. 804 East State St., Ithaca, N. Y.

JUNIORS (*Continued*)

STRANDBERG, GEORGE ROBERT. 1736 West 63d St., Seattle, Wash.

TAYLOR, SENECA VERN. Supt., Motor Trucks, United Fuel & Supply Co., 225 Marston Court, Detroit, Mich.

TUCKER, HENRY LEWIS. Bureau of Constr., Dept. of Public Works, 3519 Fifth Ave., Pittsburgh, Pa.

WILEY, HUGH LEMUEL. 402 Spalding Bldg., Portland, Ore.

FELLOWS

MEYER, HENRY CODDINGTON. 101 Park Ave., New York City.

REINSTATEMENT

MEMBER	Date of Reinstatement.
AERTSEN, GUILLIAEM	Feb. 5, 1915

DEATHS

FITTING, HAROLD HANSEN. Elected Junior, May 2d, 1911; Associate Member, September 3d, 1913; died January 7th, 1915.

MARVIN, FRANK OLIN. Elected Member, May 5th, 1897; died February 6th, 1915.

Total Membership of the Society, March 4th, 1915, 7 721.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(February 2d to March 1st, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Steel and Iron,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 20c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung,* Leipzig, Germany.

LIST OF ARTICLES

Bridges.

Report of Committee 7, Am. Ry. Eng. Assoc., on Wooden Bridges and Trestles. **(85)** Vol. 15.

Secondary Stresses.* (Appendix C, Report of Committee 15, Am. Ry. Eng. Assoc.) **(85)** Vol. 15.

Bridge Clearance Diagram. (Appendix E, Report of Committee 15, Am. Ry. Eng. Assoc.) **(85)** Vol. 15.

Requirements for the Protection of Traffic at Movable Bridges. (Appendix D, Report of Committee 15, Am. Ry. Eng. Assoc.) **(85)** Vol. 15.

Rolling Loads on Bridges.* J. E. Greiner. **(85)** Vol. 15.

Concerning Railroad Bridges Movable in a Vertical Plane.* B. R. Leffler. **(85)** Vol. 15.

A Plea for Beautiful Bridges. H. G. Tyrrell. (Paper read before the Oregon Soc. of Engrs.) **(1)** Jan.

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Bridges—(Continued).

Progress of the New Quebec Bridge.* (12) Jan. 29.
Cost of Drilling Anchor-Bolt Holes for the Quebec Bridge.* C. C. Phelps. (13) Feb. 4.
Progress on Summit Cut-Off of the Lackawanna.* (15) Feb. 5.
Five Bridges Erected in Two Days.* (14) Feb. 6.
Half-Mile Concrete Viaduct Provides Double-Deck Trafficway in Kansas City.* H. H. Fox. (14) Feb. 6.
Design Features of the Various Types of Reinforced Concrete Bridges Along the Columbia Highway in Oregon.* K. P. Billner. (86) Feb. 10.
Estimating Curves for Standard Bridges of the Illinois Highway Department.* (86) Feb. 10.
Electrical Control-Gear for Dalmuir Swing-Bridge Over Canal.* (11) Feb. 11.
Wind Stresses in Railroad Bridges. R. Fleming. (13) Feb. 11.
Kilburn Road Bridge, Metropolitan Railway.* (23) Feb. 12.
Mission Street Viaduct.* (13) Feb. 18.
Raising a Highway Bridge by End-Pin Slings.* R. C. Hardman. (13) Feb. 18.
Design, Construction and Detailed Costs of the McKinley Ford Bridge, La Salle County, Illinois.* (86) Feb. 24.
Wind Stresses in Highway Bridges. R. Fleming. (13) Feb. 25.
Equivalent Uniform Loads for Long-Span Bridges. D. B. Steinman. (13) Feb. 25.
Méthode Expéditive pour le Calcul des Voutes.* Mesnager. (33) Feb. 6.
Der steife Bogen oder Rahmen als Sonderfall des geschlossenen steifen Ringes.* J. Melan. (53) Nov. 27.
Umbau der Eisenbahnbrücke über die Wupper in Elberfeld-Sonnborn.* Stephani. (40) Dec. 19.
Die Ufersicherungen unter der Levensauer Hochbrücke bei der Erweiterung des Kaiser-Wilhelms-Kanal.* Walter Menningen. (40) Serial beginning Dec. 23.
Die Neckarbrücke Ziegelhausen-Schlierbach.* Albert Haug. (51) Serial beginning Jan. 21.
Brücke über die Gottleubamündung.* Stecher. (78) Feb. 3.

Electrical.

Service and Tests of Mayari Incline Cables.* James E. Little. (98) Dec.
Electric Light, A Factor in Civilization.* S. E. Doane. (4) Jan.
A New Method for the Measurement of the Logarithmic Decrement of Condensers.* August Hund. (73) Jan. 8.
Electric Accumulators and Their Application to Automobile Traction. R. T. Mitchell. (Paper read before the Assoc. of Engrs.-in-Charge.) (73) Jan. 22.
Modern Electric Lighting. H. C. Wheat. (Paper read before the Rugby Eng. Soc.) (47) Jan. 22.
The Testing of Potentiometers. Frank Wenner and Ernest Weibel. (Abstract of paper from *Bulletin*, Bureau of Standards.) (73) Jan. 29.
Overload Protection on Alternating-Current Circuits by Tripping Devices.* C. C. Garrard. (73) Serial beginning Jan. 29.
Power Circuit Interference with Telegraphs and Telephones.* S. C. Bartholomew. (Abstract of paper read before the Institution of Post Office Elec. Engrs.) (73) Jan. 29.
The Design and Operation of the Cleveland Municipal Electric Plant.* Frederick W. Ballard. (55) Feb.
Distortion of Alternating Current Wave Caused by Cyclic Variation in Resistance.* Frederick Bedell and E. C. Mayer. (42) Feb.
The Characteristics of Electric Motors Involved in Their Application. D. B. Rushmore. (42) Feb.
Searchlights.* C. S. McDowell. (42) Feb.
A 100 000-Volt Portable Substation.* Charles I. Burkholder and Nicholas Stahl. (42) Feb.
Dimmers for Tungsten Lamps.* Alfred E. Waller. (42) Feb.
Effect of Moisture in the Earth on Temperature of Underground Cables.* L. E. Imlay. (42) Feb.
Comparison of Calculated and Measured Corona Loss Curves.* F. W. Peek, Jr. (42) Feb.
Electrical Precipitation Theory of the Removal of Suspended Matter from Fluids. W. W. Strong. (42) Feb.
Oil Circuit Breakers, Notes on Arc Phenomena and Tendencies in Design.* K. C. Randall. (42) Feb.
Concrete Trolley Poles.* (67) Feb.
The Magnetization of Iron at High Flux Density with Alternating Currents.* J. S. Nicholson. (77) Feb. 1.
The Shape of the Pressure Wave in Electrical Machinery.* S. P. Smith and R. S. H. Boulding. (77) Feb. 1.
The Current Transformer.* A. G. L. McNaughton. (77) Feb. 1.
The Constant-Current Transformer.* John A. Randolph. (64) Feb. 2.

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A New Nitrogen Electric Furnace.* E. Kilburn Scott. (Paper read before the Soc. of Chemical Industry.) (26) Feb. 5.
The London & District Electricity Supply Bill. (73) Serial beginning Feb. 5.
Protective Switchgear.* C. Jones. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Feb. 5.
Across the Continent by Telephone.* (46) Feb. 6.
Inside Wiring Methods.* John Carrell. (111) Serial beginning Feb. 6.
Small-Town Distribution and Management Cost.* (27) Feb. 6.
Concrete Transmission-Line Poles.* R. D. Coombs. (27) Feb. 6.
Seattle Municipal Lighting Plant.* W. L. Kidston. (64) Feb. 9.
The Current Transformer.* M. Rosenbaum. (73) Feb. 12.
Protective Devices Against Lightning and Surges.* E. Kilburn Scott and L. F. Fogarty. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Serial beginning Feb. 12.
Sporadic Insulator Troubles.* P. M. Downing. (111) Feb. 13.
Illumination of Panama-Pacific Exposition.* G. L. Bayley. (27) Feb. 13.
Two American-Built Locomobile Power Plants.* (27) Feb. 13.
Proportioning of Railway Motor Resistances.* A. M. Buck. (17) Feb. 13.
Separation of the No-Load Stray Losses in a Continuous-Current Machine by Stroboscopic Running-Down Methods. David Robertson. (77) Feb. 15.
Kalamazoo Municipal Plant (for Street Lighting).* Thomas Wilson. (64) Feb. 16.
A Gas-Tractor Power Plant.* C. V. Hull. (64) Feb. 16.
Methods and Costs of Electric Shovel Work Removing Slides and Side Cutting for Electric Railway.* (From the *Excavating Engineer.*) (86) Feb. 17.
Municipal Electric-Lighting Plants in California. (13) Feb. 18.
The Ultrandion Detector for Undamped Waves.* Lee de Forest. (27) Feb. 20.
Electric-Steam Tunnel Crane.* (18) Feb. 27.
Central-Station Development at Portland, Maine.* (27) Feb. 27.
Calculation of Electromagnet Windings.* E. E. George and Harold Pender. (27) Feb. 27.
Les Installations de la Compagnie Parisienne de Distribution d'Electricité. Edouard Imbs. (32) June.
Die elektrische Kraft-und Licht-Betrieb in der Hauptwerkstätte Danzig.* Crayen. (102) Dec. 1.
Riehenschaltungs-Signallampen. L. Bloch. (41) Jan. 21.
Eine gefahrlose metallische Röntgenrohre.* L. Zehnder. (41) Feb. 4.

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The Geared Turbine Machinery of the *Transylvania.** (11) Serial beginning Jan. 29.
The Works of Canadian Vickers, Limited, at Montreal.* (11) Feb. 5.
Fog and Fog Signals.* Edwin O. Catford. (12) Feb. 5.
The Anchor Liner *Tuscania.** (11) Feb. 12.
A New Car Ferry with Adjustable Deck.* (15) Feb. 26.
Le Ravitaillement en Mer des Navires de Guerre.* (33) Feb. 13.

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The Theory of Hardening and the Constitution of Steel. Edward D. Campbell. (71) Vol. 90.
The Bye-Product Coking Industry and Its Relation to the Manufacture of Iron and Steel.* G. Stanley Cooper. (71) Vol. 90.
Influence of Riveting on the Stresses in the Rivet and on the Strength Properties of the Rivet Material. Hans Rudeloff. (71a) Vol. 6.
Modern Bye-Product Coking.* G. Stanley Cooper. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 48, Pt. 2.
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The Removal of Carbon Bisulphide from Coal Gas by Heat. E. V. Evans. (Paper read before the Soc. of Chemical Industry.) (66) Jan. 26.
Aerial Navigation.* R. T. Glazebrook. (Paper read before the Royal Institution.) (11) Serial beginning Jan. 29.
Essential Principles of Engine Design; Also Materials of Construction. Frank Foster. (Paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 29.
A Big Consumer's Storage Yard, New York Edison Co.'s Plant at Shadyside, N. J.* J. F. Springer. (45) Feb.
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The New Keilehaven Gas-Works at Rotterdam.* M. C. Sissingh. (Paper read before the Royal Dutch Inst. of Engrs.) (66) Feb. 2.
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The Jitney Bus. Charles N. Black. (111) Feb. 13.
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Pittsburgh's Progress in Smoke Abatement.* J. W. Henderson. (62) Feb. 15.
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Recent Gas Piping Practice in Philadelphia. H. R. Sterrett. (Paper read before the Illuminating Eng. Soc.) (83) Feb. 15.
Suiting the Stoker Design to the Plant.* T. A. Peebles. (62) Feb. 15.
Thirteen Years' Operation of Inclined Retorts.* Frank Huber. (Paper read before the A. G. I. Assoc.) (24) Feb. 15.
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The Briquetting of Saskatchewan Lignite.* (96) Feb. 18.
The Rolling Mill as a Machine-Shop Product.* (72) Feb. 18.
Safety Derrick-Hook for Handling Buckets.* N. R. Melvin. (13) Feb. 18.
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Oxyacetylene Welding in Mining.* (16) Feb. 27.
The Demand Rate for Gas Service. C. H. Cook. (Paper read before the Wisconsin Gas Assoc.) (83) Mar. 1.
New Coal Gas Plant at Fall River.* C. W. Hunter. (Paper read before the New England Assoc. of Gas Engrs.) (83) Mar. 1.
Ueber neuere Gasreinigungsverfahren.* H. Strache. (53) July 31.
Ist die Berechnung von Dampfleitungen nach Prof. Rietschels Leitfaden zuverlässig? H. Roose. (7) Dec. 12.
Die neue Automobil-Ausstellungshalle am Kaiserdamm in Berlin.* Hans Schmuckler. (48) Jan. 16.
Die Grossdieselmotorschiffe, ihre Wirtschaftlichkeit und ihre Zukunft. Wm. Scholz. (48) Jan. 30.

Metallurgical.

The Oxygen Content of Open Hearth Steel. J. Allen Pickard and F. M. Potter. (71) Vol. 90.
The Determination of Cobalt in High-Speed Steels. Lawrence Dufty. (71) Vol. 90.
The Bye-Product Coking Industry and Its Relation to the Manufacture of Iron and Steel.* G. Stanley Cooper. (71) Vol. 90.
Note on the Transformations of Steels.* H. De Nolly and L. Veyret. (71) Vol. 90.
Mechanical Charging of Blast Furnaces.* N. Kapp. (71) Vol. 90.
Electrolytic Iron, Its Manufacture, Properties, and Uses.* L. Guillet. (71) Vol. 90.
Utilisation of Heat Contained in Slag.* Walter L. Johnson. (71) Vol. 90.
A New Process for Heating Blast Furnace Stoves.* A. Spannagel. (71) Vol. 90.
The Use of Liquid Ferro-Manganese in the Steel Processes.* Axel Sahlin. (71) Vol. 90.
The Influence of Coalescence on the Mechanical Properties of Steel and on Alloys.* A. M. Portevin and V. Bernard. (71) Vol. 90.
The Decarburisation of Steels in the Salt Baths Used for Heating Prior to Hardening. A. M. Portevin. (71) Vol. 90.
Methods of Transport of Raw Material in the Iron Industry.* J. F. Apfelstedt. (71*a*) Vol. 6.
Electric Furnaces for Heating Steel.* Alcan Hirsch. (71*a*) Vol. 6.
The Gases Occluded in Liquid Steel. L. Baraduc-Muller. (71*a*) Vol. 6.
An Examination of Fire Bricks and Some Other Technical Refractory Materials.* W. Hamilton Patterson. (71*a*) Vol. 6.

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The Influence of Slag and Fumes on the Zinc Muffle. O. Proske. (Translated from *Metall und Erz*, by F. Sommer Schmidt.) (103) Jan. 30.
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The Case for the Electric Lamp. William Maurice. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 48, Pt. 2.
Coal-Mining in the State of Pennsylvania, United States of America.* Samuel Dean. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 48, Pt. 2.
A Firedamp Indicator.* H. R. Webster. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 48, Pt. 2.
The Killingworth Colliery (New South Wales) Explosion.* James Ashworth. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 48, Pt. 2.
The Maikop Oilfield, South Russia.* William Calder. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 48, Pt. 2.
Substitutes for Wooden Supports of the Roof in Longwall Working. W. Hutton Hepplewhite. (Paper read before the Midland Counties Institution of Engrs.) (106) Vol. 48, Pt. 2.
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A Novel Plan for Stopping a Landslide at Mount Vernon.* N. H. Darton. (13) Feb. 25.

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Snow Removal, A Report of the Committee on Resolutions of the Snow Removal Conference Held in Philadelphia, April 16 and 17, 1914. (55) Feb.
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Treated Wood Block Pavements, Their History in the United States.* (Report of Committee of the Am. Wood Preservers' Assoc.) (60) Feb.
Methods and Cost of Constructing a Concrete Road in La Salle Co., Illinois.* B. H. Piepmeir. (Paper read before the Illinois Soc. of Engrs. and Surveyors.) (86) Feb. 3.
A Modification of the New York Standard Guard Rail Providing Concrete Posts.* H. E. Smith. (86) Feb. 3.
Methods and Cost of Constructing Nine Miles of Concrete State Road in California. J. B. Woodson. (From *California State Highway Bulletin.*) (86) Feb. 3.
Dry Sand and Cement Mixture *vs.* Mortar Bed for Wood Block Pavements. Theodor S. Oxholm. (Paper read before the Am. Assoc. for the Advancement of Science.) (13) Feb. 4.
Notes on Extraordinary Traffic. F. Oscar Kirby. (Paper read before the Institution of Mun. and County Engrs.) (104) Feb. 5.
Method and Cost of Concrete Road Construction by Day Labor Under the Supervision of the Illinois Highway Commission.* A. H. Hunter. (Paper read before the Illinois Soc. of Engrs. and Surveyors.) (86) Feb. 10.
Patching Bituminous Pavements without an Asphalt Plant or a Steam Roller.* Samuel H. Lea. (18) Feb. 11.
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Some Features of Brick Road Construction in Illinois. Rodney L. Bell. (Paper read before the Illinois Soc. of Engrs. and Surveyors.) (86) Feb. 17.
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Grade Reduction Problems.* C. P. Howard. (85) Vol. 15.
Extra Top Width for New Fills.* J. C. L. Fish. (85) Vol. 15.
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Notes on L. C. L. Freight Houses.* E. H. Lee. (85) Vol. 15.
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Influence of Aluminum and Silicon on Bessemer Ingots and Rails.* M. H. Wickhorst. (Appendix F, Report of Committee 4, Am. Ry. Eng. Assoc.) (85) Vol. 15.
Seams in Rails as Developed from Cracks in the Ingot.* M. H. Wickhorst. (85) Vol. 15.
Influence of Seams or Laminations in Base of Rail on Ductility of Metal.* H. B. MacFarland. (85) Vol. 15.
Comparison of Basic and Acid Open-Hearth Rails, and Influence of Reheating Cold Blooms.* M. H. Wickhorst. (85) Vol. 15.
Rail Failure Statistics for the Year Ending October 31, 1912. R. Trimble. (Appendix A, Report of Committee 4, Am. Ry. Eng. Assoc.) (85) Vol. 15.
Influence on Rails of Amount of Draft in Blooming.* M. H. Wickhorst. (85) Vol. 15.
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Japanese Railways Dynamometer Car.* Edward C. Schmidt. (25) Feb.; (18) Jan. 6.
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Design and Construction Features of Reinforced Concrete Sewer Siphons Under New York Subways. Thad. L. Wilson. (Paper read before the Brooklyn Engrs.' Club.) (86) Feb. 10.
Design, Construction and Cost of New Sanitary Sewerage System at Alton, Ill. J. E. Schwaab. (Paper read before the Illinois Soc. of Engrs. and Surveyors.) (86) Feb. 10.
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*Illustrated.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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PAPERS AND DISCUSSIONS

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THE ST. JOHN LEVEE AND DRAINAGE DISTRICT OF MISSOURI.

By R. M. Strohl, Jun. Am. Soc. C. E.

To be Presented April 7th, 1915.

Synopsis.

In view of the fact that, at present, there is little information relating to the quantity of run-off, impounded storage, back-water curves, and other matters of direct importance to drainage districts in the Mississippi Valley, and that even these vary greatly in different localities, it is believed that a record of such features for Southeastern Missouri will be of some value; also, as little has been written concerning the assessment of benefits accruing from the construction of reclamation systems, an outline of this work, as applied to the territory included within this district, will be of value to those engaged in operations of this kind.

A record of the assumptions and data on which the work of the St. John Levee and Drainage District has been designed, compared with the actual results obtained after its completion, will give an authoritative basis for future undertakings.

This paper treats of some of the drainage problems of Southeastern Missouri, and gives in detail the data necessary to be considered in designing a system for the complete and economical reclamation of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

large tracts of overflow land. Levees, floodways, drainage ditches, floodgates, storage basins, and siphons, are considered and discussed in detail, as they are found necessary.

The assessment of benefits, as provided for under the Missouri laws, is also outlined and applied to the case, and an effort has been made to show a just and equitable distribution of the cost of the proposed work.

Location.—The St. John Levee and Drainage District of Missouri is in that part of Southeastern Missouri lying along the Mississippi River and across from Cairo, Ill. It is bounded on the north by Scott County and the Big Lake Drainage and Levee District of Missouri, on the east and south by the Mississippi River, and on the west by a low ridge known as the Sikeston Ridge. The southern portion of this ridge is included.

Soil.—The lands of the District are especially fertile, having been built up by deposition by the Mississippi River. The soil in the northern portion of the District consists largely of a sandy loam which is easily cultivated. Underlying this at no great depth is a stratum of sand. As the southern end of the District is approached, the sand stratum is found at an increasing depth, and is overlaid by finer sedimentary deposits, gumbo and clay, varying in thickness from 2 to 20 ft., or more.

Surface Features.—The surface is generally a plain with an average slope to the south of about 1 ft. per mile.

Two natural basins, locally known as Ten Mile Pond Basin and Eagle Nest Cypress Basin, are found in the southern part, and are separated by Sugar Tree Ridge, which extends northwest and southeast to a point about ¾ mile from the Mississippi River at the south central part of the District.

Natural levees have been forming and, except for two bayous, namely, the St. John Bayou and James Bayou, have shut off effectively the drainage from the interior.

There are three outlets, the two bayous just mentioned and one artificial channel, for the drainage of the interior lands. Several small bayous, namely, East, Wilson, and Black Bayous, aid in the drainage of the lands in the southern part of the District.

Area.—The District now embraces about 325 000 acres, taken up by two political subdivisions, as follows: Mississippi County, 225 000 acres, and New Madrid County, 100 000 acres.

Local Differences.—The natural organization of the District should include all of Mississippi County, that part of New Madrid County now included, and a part of Scott County.

A practice has existed throughout the Mississippi Valley whereby the high lands have effected their reclamation at the expense of lands lower in the valley, by draining their waters to the lower lands, and by not organizing for putting into effect a complete system of reclamation, but by working as individuals.

Local Districts.—Numerous drainage and levee districts are organized and have effected partly the reclamation of large areas within the District, but, for their further development, an organization of two or more counties will be necessary.

Organization of the St. John Levee and Drainage District of Missouri.

Originally, the St. John Levee and Drainage District embraced only that part of New Madrid County now within the District and east of the Sikeston Ridge, and was incorporated by decree of the New Madrid County Circuit Court on March 29th, 1912. Plans were prepared for the reclamation of the District, but they were not economical, and, for these and other reasons, the land owners of Mississippi County petitioned the Court for a change of plan and the enlargement of the District; this enlargement was effected in September, 1913.

Plans were immediately begun for the reclamation of the enlarged District, and were adopted by the District on April 15th, 1914.

Preliminary Considerations.—In making up the reclamation plan for the District, it was first necessary to investigate the systems of drainage and protection from overflow which had been constructed by the various local organizations, and determine to what extent they furnished relief to the various lands.

The present protection is afforded by a continuous levee under the control of local levee districts, namely, Levee Districts Nos. 1 and 2 of Mississippi County, Levee District No. 2 of Scott County, and the Mississippi River Commission.

This levee varies from 5 to 18 ft. in height, and has a total length of about 55 miles; it extends from what is known as Philadelphia Point, in Scott County, to Dorena, at the southern end of Mississippi County.

The grade to which this levee is built is that established by the Mississippi River Commission in 1910. All the levees are in good condition except that in Scott County, which has not been repaired since the flood of 1913.

The ditches of the various drainage districts head in the northern end of the District and in Scott County, and, with the exception of District No. 30 of Mississippi County, which has its own outlet through the levee, they flow in a southerly direction.

All but two of the ditches flowing south in Mississippi County converge and have outlet through the Lee-Rowe Ditch to the Mississippi River. The drainage from the two ditches excepted, with that of about 90 000 acres from Scott County, is brought to the St. John Bayou through dredge ditches.

The drainage of New Madrid County lands not having outlet through these ditches finds its way through Eagle Nest Cypress Basin, Wilson Bayou, and East Bayou, to St. John Bayou.

Many ditches are inefficient through lack of sufficient capacity, largely on account of sliding banks and willow growth. Approximately 187 000 acres of land within the District are subject to overflow from back-water and head-water from the Mississippi River.

Plan for Reclamation.—A plan to provide complete reclamation for the overflow lands and better protection for the high lands above overflow from back-water, must include:

1. Levees to prevent overflow and back-water from the Mississippi River;
   (*a*) Enlargement of existing levees;
   (*b*) Construction of additional levees with flood-gates at St. John Bayou;
2. Drainage ditches and pumping plants to dispose of the surplus rain-water falling on these lands;
   (*a*) To collect the waters from all the lands above back-water overflow and conduct them through a high-level ditch to the Mississippi River;

(*b*) To conduct through a low-level diversion ditch, the remainder of the surface water which would flow out at James Bayou and the Lee-Rowe Ditch, to St. John Bayou, so as to permit the closing of these outlets, and thus lower the head of the back-water;

(*c*) Collection ditches on the lands below overflow and a pumping plant to take care of the surplus water during periods of overflow in the Mississippi River or when the flood-gates must be closed.

Conditions can be greatly improved in two or three of the existing ditches. This, however, will not be taken up in this plan, but will be left for improvement as the land owners along each ditch may desire.

District Boundaries.—It requires only a casual examination to disclose the fact that in order to work out a complete and economical project, all the territory south of the Commerce Hills and between the Sikeston Ridge and the Mississippi River must be considered; and the District should be enlarged to include all territory not now within the boundaries of the St. John Levee and Drainage District of Missouri.

In addition, the proposed improvements will affect materially that portion of the Little River Valley which is subject to severe overflow from waters that pass across the Sikeston Ridge in the vicinity of New Madrid; and the District should be enlarged to include the lands lying between the St. John Levee and Drainage District and the Little River Drainage District.

Levee Protection.—As nearly all the lands within the District and those which should be included are subject to overflow by the Mississippi River, it is necessary to provide a levee sufficiently high and strong to confine these waters to their natural channel.

Investigations were made to determine the elevation of the flood planes of the various overflows. Since the overflow of 1913, it has been necessary to provide protection against floods of equal volume and height, and as the Mississippi River Commission grade of 1910 was no longer considered safe, a new grade was decided on for this work, and, later, a grade was established by the Mississippi River Commission, which would be of sufficient height to confine the volume of the flood of 1912.

The junction of the Ohio and Mississippi Rivers brings about a difficult condition, and, at times of flood, the effect of their confluence can be readily seen. Fixing a grade at from 3 to 4 ft. above the 1913 flood-plane calls for an unbroken levee from Commerce to New Madrid. At present there is no levee from Commerce to Philadelphia Point in Scott County, nor from Dorena, in Mississippi County, to the high land above the overflow about 1 mile below New Madrid.

The existing levee is to be used where it can be enlarged satisfactorily. This enlargement can be effected on all except about 1½ miles of poorly located levee in Scott County. The levee will be brought to the standard of the Mississippi River Commission throughout, namely, an 8-ft. crown with 3:1 side slopes and a 4-ft. muck-ditch, which it has fixed for this river district.

At the crossing of the levee and James Bayou and the Lee-Rowe Ditch, the section will be increased to an 8-ft. crown with 4:1 side slopes. A 40-ft. banquette, beginning 10 ft. below the crown, is to be added on the inside. One row of sheet-piling will be placed near the center of the levee in order to cut off seepage.

A special levee has been designed for the water-front at New Madrid. To protect the property of the city, the levee must be as close to the river as possible, where the bank is protected by revetment placed about 15 years ago. In order that the property damage might be limited and few removals of buildings be made necessary, a broad, flat levee with a crown width of 10 ft. and 10:1 side slopes was decided on. Buildings and dwellings on the site are to be raised on their present locations above levee grade, and the material for the levee is to be pumped in from the river by a suction dredge.

Drainage Outlets Through the Levee.—There will be three openings through the levee, two of which have already been provided for by local organizations, namely, the Big Lake Drainage and Levee District of Missouri and District No. 30 of Mississippi County, Missouri, the third being the outlet of St. John Bayou under this plan.

Design of Outlets.—The accompanying drawings, Fig. 1 and Plates II to V, show the general plan of outlet, as designed by the writer, for District No. 30. This outlet is placed in what is known as P. Field's Bayou. The gate and hoist have been furnished by the Coffin Valve Company.

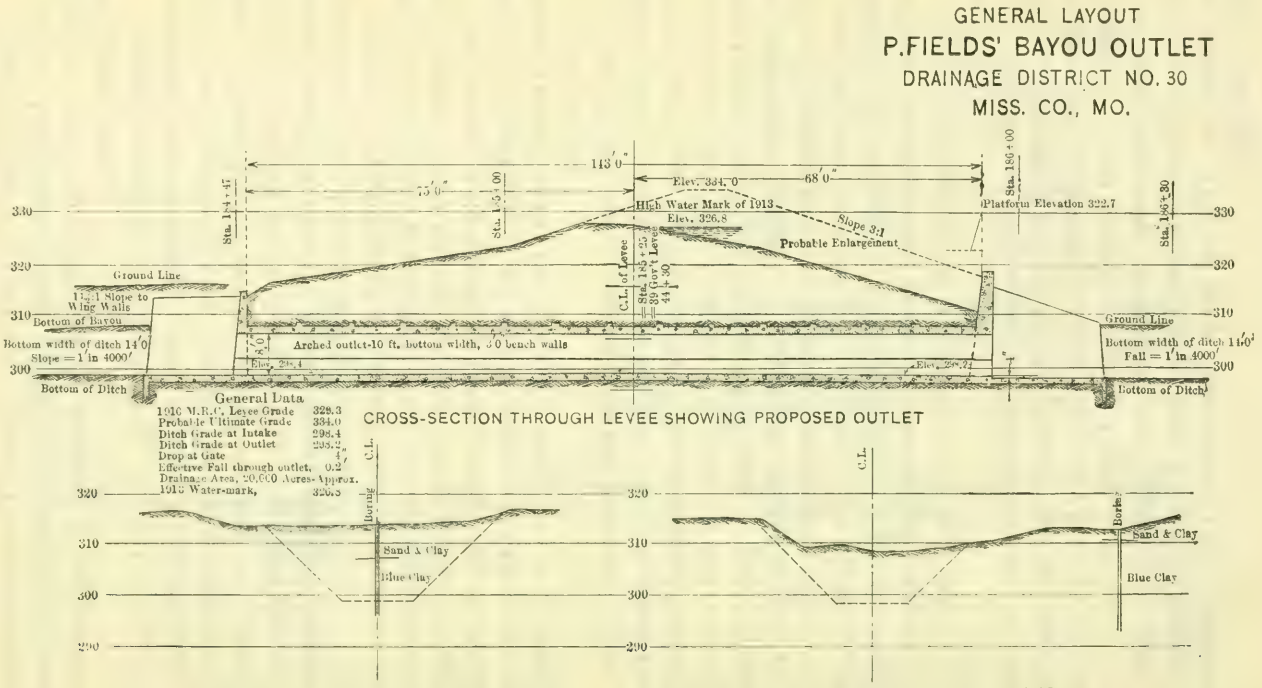

GENERAL LAYOUT
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.
143' 0"
75' 0"
68' 0"
Elev. 334.0
High Water Mark of 1913
Elev. 326.8
Slope 3:1
Probable Enlargement
Platform Elevation 322.7
Sta. 184+47
Sta. 185+00
Sta. 186+00
Sta. 186+30
Ground Line
1½:1 Slope to Wing Walls
Bottom of Bayou
Bottom width of ditch 14'0"
Slope = 1' in 4000'
Arched outlet-10 ft. bottom width, 3'0 bench walls
Elev. 298.4
Elev. 298.71
Ground Line
Bottom width of ditch 14'0"
Fall = 1' in 4000'
Bottom of Ditch
Bottom of Ditch
C.L. of Levee
Sta. 185+55
39' 0" to Levee
41' 30"
General Data
1916 M.R.C. Levee Grade 328.3
Probable Ultimate Grade 334.0
Ditch Grade at Intake 296.4
Ditch Grade at Outlet 295.2
Drop at Gate 1'
Effective Fall through outlet, 0.2'
Drainage Area, 20,000 Acres-Approx.
1913 Water-mark, 326.8
CROSS-SECTION THROUGH LEVEE SHOWING PROPOSED OUTLET
C.L.
Boring
Sand & Clay
Blue Clay
CROSS-SECTION OF DITCH AT STA. 184+47
C.L.
Boring
Sand & Clay
Blue Clay
CROSS-SECTION OF DITCH AT STA. 186+30

FIG. 1.

Design of St. John Bayou Outlet.—Plate VI shows the general plan of the St. John Bayou Outlet. Details of the gates, gate-frames, and hoists will be in conformity with the standards of the manufacturers who furnish them.

Foundation conditions for the St. John Bayou Outlet, as shown by borings, will necessitate strong coffer-dam construction, steel sheet-piling being deemed best for the work. An open coffer-dam with inside dimensions of 110 by 150 ft., and piling from 24 to 30 ft. in length, will be required. It will also be desirable to have the coffer-dam free from bracing, in order to allow the work to proceed without obstruction, and a plan to this effect has been developed.

Drainage.—Practically all the run-off from the lands of the District, as well as from about 90 000 acres in Scott County, passes into the Mississippi River through the three outlets previously mentioned.

The cutting of the Lee-Rowe Ditch directly into the Mississippi River was ill-advised, as an efficient outlet could have been secured through James Bayou at less cost, and it would not now be necessary to close this opening.

Of the 415 000 acres draining into the Mississippi River, about 190 000 acres are still subject to overflow from back-water from the river at flood stages like that of 1913. This leaves about 225 000 acres that will drain by gravity to the river at a stage of 44.6 ft. on the New Madrid gauge, which was the high-water mark of the 1913 flood and was never before equalled; it corresponds to an elevation of about 307.19 ft., Memphis datum, to which all work in this district is referred.

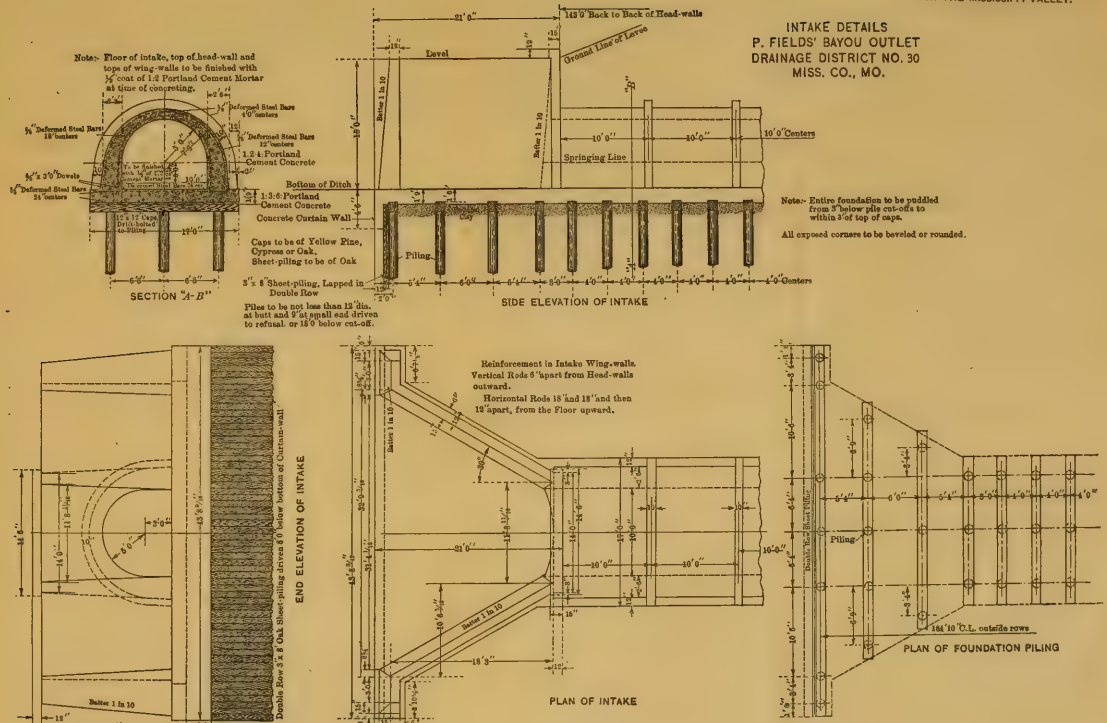
With the building of the levee along the river and the consequent closing of the drainage outlets, it is seen that unless the water from the lands above back-water overflow is provided for separately, it will be a source of damage and a burden to the lands subject to such overflow.

To meet this, an intercepting or diversion floodway, or high-level ditch, was planned. This would take the waters from the several ditches and carry them to an outlet into the Mississippi River at New Madrid at stages up to and equivalent to that of 1913.

To collect the waters below back-water overflow and divert them from the Lee-Rowe Ditch and James Bayou, a low-level diversion ditch was decided on. This will carry these waters to the outlet provided for St. John Bayou.

PLATE II.
PAPERS, AM. SOC. C. E.
MARCH, 1915.
STROHL ON
DRAINAGE PROBLEMS
IN THE MISSISSIPPI VALLEY.

INTAKE DETAILS
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.


Note:- Floor of intake, top of head-wall and tops of wing-walls to be finished with ½" coat of 1:2 Portland Cement Mortar at time of concreting.
Deformed Steel Bars 12" centers
Deformed Steel Bars 24" centers
1:2:4 Portland Cement Concrete
Bottom of Ditch
1:3:6 Portland Cement Concrete
Concrete Curtain Wall
Caps to be of Yellow Pine, Cypress or Oak. Sheet-piling to be of Oak
3"x 8" Sheet-piling, Lapped in Double Row
Piles to be not less than 12" dia. at butt and 9" at spall end driven to refusal, or 18'0" below cut-off.
SECTION "A-B"
21'0"
143'0" Back to Back of Head-walls
Ground Line of Levee
Level
Batter 1 in 10
Springing Line
10'0" Centers
Piling
Note:- Entire foundation to be puddled from 3' below pile cut-offs to within 3' of top of caps.
All exposed corners to be beveled or rounded.
SIDE ELEVATION OF INTAKE
END ELEVATION OF INTAKE
Double Row 3"x 8" Oak Sheet-piling driven 8'0" below bottom of Curtain-wall
Reinforcement in Intake Wing-walls.
Vertical Rods 6" apart from Head-walls outward.
Horizontal Rods 18" and 18" and then 12" apart, from the Floor upward.
PLAN OF INTAKE
PLAN OF FOUNDATION PILING
164'10" O.L. outside rows

Design.—In the design of drainage projects, run-off is an uncertain factor, and although many formulas have been worked out, they are more or less guesswork, vary greatly for different localities, and will not be the same in any one locality throughout the year. It is best to take advantage of the experience of districts adjoining or included in those under consideration.

Measurements made to determine run-off in the Little River Drainage District of Missouri which is just west of the St. John Levee and Drainage District, show that, for areas where there is no hill run-off, it is less than $\frac{1}{4}$ in. in 24 hours. Ditches in the various districts of Mississippi County which were designed to care for a run-off of $\frac{1}{4}$ in. in 24 hours have never overflowed their banks.

The lands at the northern end of the District, being largely composed of sand or having a sandy sub-soil, have a great capacity for absorbing rain water. No hill run-off has to be provided for, so it was decided to design the ditches for the following quantities:

Areas up to 200 sq. miles..... $\frac{1}{4}$ in. in 24 hours.
Areas from 200 to 300 sq. miles..... $\frac{7}{32}$ in. in 24 hours.
Areas of more than 300 sq. miles..... $\frac{3}{16}$ in. in 24 hours.

The High-Level Ditch or Floodway.—To facilitate the design of this ditch, it is divided into sections which are marked by its junctions with the intercepted ditches, and each section is designed separately. The main ditch has its head in the ditch farthest east in Mississippi County, and the secondary ditch has its head in the one nearest the Sikeston Ridge.

The approximate areas and discharges of each section were ascertained, and are given in Tables 1 to 5, inclusive.

TABLE 1.—HIGH-LEVEL DITCH (MAIN).

Section.	Area, in acres.	Run-off, in inches.	Discharge, in second-feet.
1	30 000	1/4	315.0
2	10 120	1/4	106.2
3	31 300	1/4	328.6
4	4 120	1/4	43.2
5	13 400	1/4	140.7
6	11 680	1/4	122.6
7	250 000	3/16	1 975.0

TABLE 2.—Summary of Discharge from Sections, in Second-Feet (Main).

Section.	Discharge.	Increments.				
1	315.0					
2	421.2	106.2				
3	749.8	434.8	328.6			
4	793.0	478.0	371.8	43.2		
5	933.7	618.7	512.5	183.9	140.7	
6	1 056.5	741.3	635.1	306.5	263.3	122.6
7	1 975.0	(Main ditch below junction.)				

TABLE 3.—High-Level Ditch (Secondary).

Section.	Area, in acres.	Run-off, in inches.	Discharge, in second-feet.
1	35 340	¼	371.1
2	70 000	¼	735.0
3	12 200	¼	128.1
4	4 500	¼	47.2
5	23 000	¼	241.5

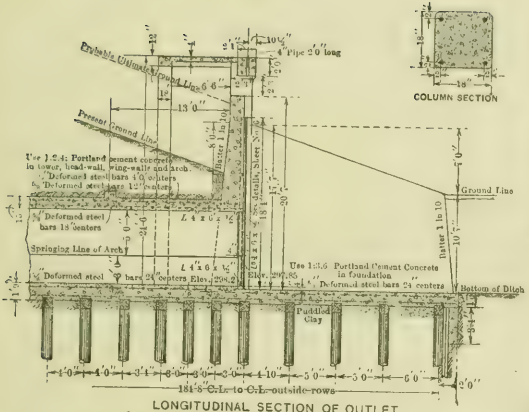
TABLE 4.—Summary of Discharge from Sections, in Second-Feet (Secondary).

Section.	Discharge.	Increments.	
1	1 106.1		
2	1 234.2	128.1	
3	1 281.4	175.3	47.2
4	1 334.0	(145 000 acres with $\frac{7}{32}$-in. run-off.)	

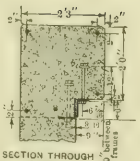
As the elevation of the grade of the bottom of the high-level ditch is fixed by the bottom of the ditch where it heads, and as it flows across the natural slope of the country, it is necessary to use very flat slopes.

In order that the lands above back-water overflow may have an outlet at flood stages of the Mississippi River and that the lower lands will not overflow, the banks of the high-level ditch must be leveed. The grade of these levees is not less than 3 ft. above the 1913 flood-plane. In construction, it is planned to use a drag-line excavator on each bank, and build the levees with the excavated material at one operation.

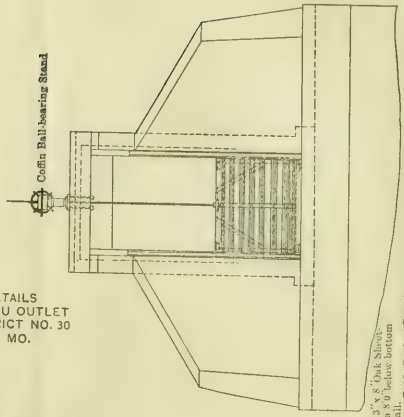
Flood Flow.—To produce a flow through the ditch at flood stages, the incoming waters must rise above the flood-plane of the river until a sufficient fall is secured to produce flow through the ditch.

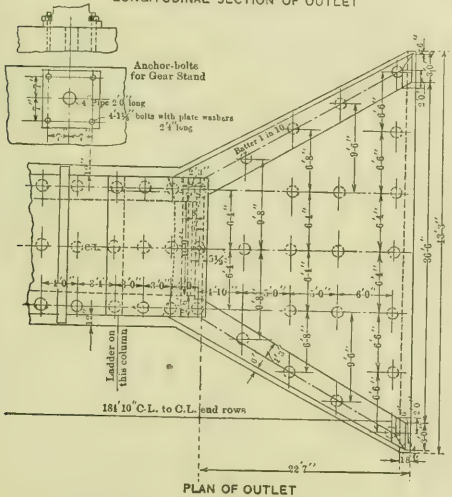

Pipe 2'0" long
Present Ground Line
Batter 1 in 10
Use 1:2:4 Portland cement concrete in tower, head-wall, wing-walls and arch
Deformed steel bars 12" centers
Deformed steel bars 15" centers
Springing Line of Arch
Deformed steel bars 24" centers
Ground Line
Use 1:3:6 Portland Cement Concrete in foundation
Deformed steel bars 24" centers
Bottom of Ditch
Puddled Clay
Elev. 297.85
Elev. 296.2
184'8" C.L. to C.L. outside rows

LONGITUDINAL SECTION OF OUTLET

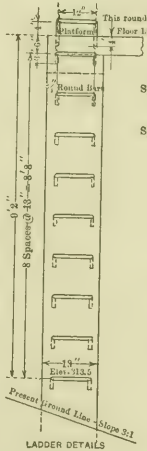


OUTLET DETAILS
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.


Coffin Ball-bearing Stand
END ELEVATION OF OUTLET SHOWING GATE IN POSITION
Double row 3" × 8" Oak Sheet piling driven 8'0 below bottom of curtain wall.

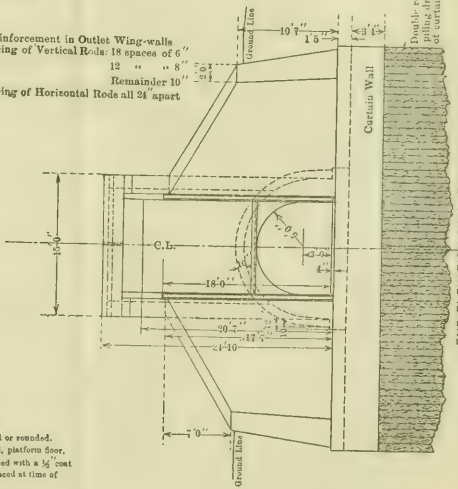

Anchor-bolts for Gear Stand
4 Pipe 2'0" long
4-1½" bolts with plate washers 2'4" long
Batter 1 in 10
Ladder on this column
184'10" C.L. to C.L. end rows
22'7"

PLAN OF OUTLET



LADDER DETAILS

Reinforcement in Outlet Wing-walls
Spacing of Vertical Rods: 18 spaces of 6"
12 " " 8"
Remainder 10"
Spacing of Horizontal Rods all 24" apart

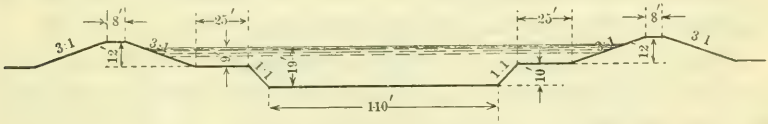


Note:- All exposed corners to be beveled or rounded. Floor of outlet, top of head-wall, platform floor, and top of wing walls to be finished with a ½" coat of 1:2 Portland cement mortar placed at time of concreting.

TABLE 5.—DESIGN OF SYSTEM.

Section No.	Estimated discharge, in second-feet.	Slope.	Bottom width, in feet.	Depth of water, in feet.	Area, in square feet.	Hydraulic radius.	Velocity, in feet per second.	Theoretical discharge, in second-feet.	Kutter's coefficient.
MAIN DITCH.									
1	315.0	0.0001	35	5.4	218.16	4.34	1.60	349.0	0.025
2	421.1	0.00008	40	6.0	276.0	4.84	1.75	483.0	0.025
3	749.8	0.00008	55	6.0	366.0	5.08	2.00	732.0	0.0225
4	793.0	0.00008	60	6.0	396.0	5.14	2.00	792.0	0.0225
5	933.7	0.00008	65	7.0	504.0	5.94	2.05	1 033.0	0.0225
6	1 956.3	0.0001	70	7.0	539.0	6.03	2.24	1 207.0	0.0225
7	1 975.0	0.0001	110	8.0	944.0	7.12	2.50	2 360.0	0.0225
SECONDARY DITCH.									
1	1 106.1	0.0001	70	7.0	539.0	6.03	2.24	1 207.0	0.0225
2	1 234.2	0.0001	75	7.0	574.0	6.05	2.24	1 286.0	0.0225
3	1 281.4	0.0001	78	7.0	595.0	6.08	2.26	1 345.0	0.0225
4	1 334.0	0.0001	80	7.5	656.0	6.48	2.37	1 555.0	0.0225

The length of the ditch is approximately 25 miles, and it is estimated that a surface fall of 0.025 ft. per mile will be sufficient to produce the required discharge. Fig. 2 shows its cross-section. Its area is 3 063 sq. ft., its perimeter, 245.16 ft., and its hydraulic radius, 12.5. Wheeler's formula, $V = 0.8\sqrt{rs}$, gives a velocity of 0.63 ft. per sec. and a discharge of 1 930 sec-ft. The velocity by Kutter's formula is somewhat greater than 0.63 ft. per sec., therefore it is assumed that this is a safe margin.



CROSS-SECTION OF MAIN DITCH

FIG. 2.

By adding the difference in elevation of the surface between the source and the outlet, to the elevation of the 1913 flood plane at New Madrid, the approximate elevation of the location was ascertained, and the ditch was located along these lines, neglecting small irregularities of the ground.

Siphon Outlet for St. John Flats.—Along the western side of the District there are very low lands, known as St. John Flats. Through this territory a dredge ditch, called St. John Ditch, or Main Ditch,

District No. 5, has been cut. It is necessary to intercept this ditch near the northwest corner of the District and to cross it with the high-level ditch near its outlet into St. John Bayou. This leaves a basin between the Sikeston Ridge and the high-level ditch without an outlet and below back-water overflow.

An inverted siphon was then planned at the crossing of the high-level ditch to transfer the waters from the basin to the outlet as before. The siphon had to be arranged so as to permit the silt that will collect therein to be easily removed, and, to this end, it was decided to use a gate to close the siphon at the upper end. A bulkhead can then be placed at the outlet of the siphon and the water and silt removed. This gate will also act as a protection to the basin in case the river-front levee should break.

Spillways.—At points where the bottom of an intercepted ditch is more than a few feet above the grade of the high-level ditch, concrete spillways or drops must be provided in order to prevent the erosion of the bottom of the ditches and the consequent silting of the high-level ditch.

At the mouth of the high-level ditch, there must be a spillway providing a drop of about 12 ft., in order to prevent the erosion of the bottom of the ditch, thus endangering the side levees.

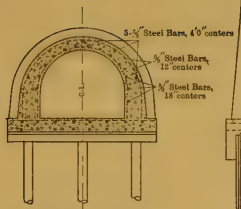
Low-Level Diversion Ditch.—This ditch, planned to divert the waters from James Bayou and the Lee-Rowe Ditch to the St. John Bayou, also crosses the natural slope of the land, and, in locating it, natural water-courses and low lands were followed as much as possible, thus effecting an appreciable saving of necessary work.

Design.—In designing this ditch, three sections were deemed sufficient, and it was planned large enough only to take the run-off from the lands tributary to it after the construction of the high-level ditch.

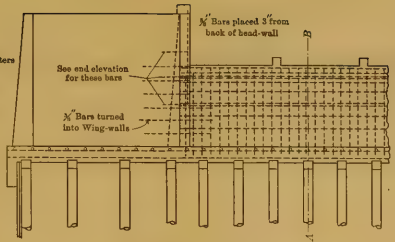
TABLE 6.—Low-Level Ditch.

Section.	Area, in acres.	Run-off, in inches.	Discharge, in second-feet.
1	26 820	¼	282.0
2	43 960	¼	461.6
3	66 320	¼	096.4

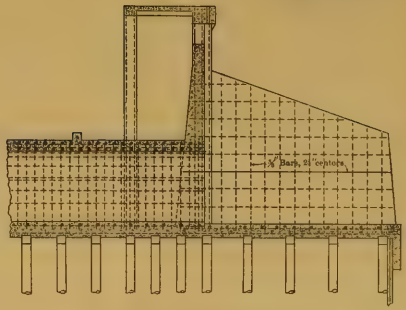
Sub-Surface Flow.—It was noted, toward the end of the long dry season of 1913, that considerable water still flowed in several ditches.



SECTION OF ARCH
SECTION *A-B*



SIDE ELEVATION AT INTAKE



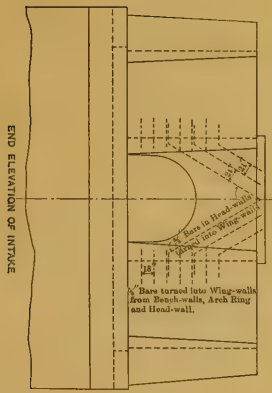
LONGITUDINAL SECTION AT OUTLET

Note:-Where ⅝" Bars are specified in the Reinforcing, these shall be ⅝" Deformed Steel Bars of approved type.

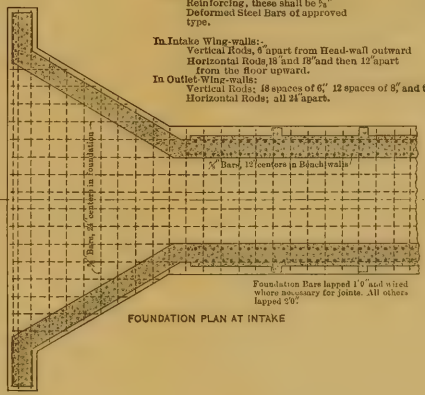
In Intake Wing-walls:-
Vertical Rods, 6" apart from Head-wall outward
Horizontal Rods, 18" and 18" and then 12" apart from the floor upward.

In Outlet Wing-walls:
Vertical Rods; 18 spaces of 6", 12 spaces of 8", and the remainder 10" apart.
Horizontal Rods; all 24" apart.

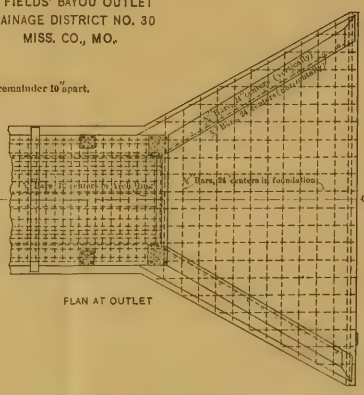
REINFORCING DIAGRAM
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.


C L
⅝" Bars in Head-wall
⅝" Bars turned into Wing-walls
18"
⅝" Bars turned into Wing-walls from Bench-walls, Arch Ring and Head-wall.

END ELEVATION OF INTAKE


⅝" Bars, 24" centers in foundation
⅝" Bars, 12" centers in Bench-walls
Foundation Bars lapped 1'0" and wired where necessary for joints. All others lapped 0'0".

FOUNDATION PLAN AT INTAKE



PLAN AT OUTLET

During September, 1913, about the end of the dry season, discharge observations were made in the St. John Ditch at three points:

(1) At its crossing with the Iron Mountain Railway;
(2) Near its outlet in St. John Bayou; and
(3) At the mouth of St. John Bayou.

TABLE 7.—Summary of Discharge from Sections, in Second-Feet.

Section.	Discharge, in second-feet.	Increment.
1	282.0	
2	743.6	461.6
3	1 100.0	137 100 acres at $\frac{7}{32}$-in. run-off.

TABLE 8.—Design of Low-Level Ditch.

Section No.	Estimated discharge, in second-feet.	Slope.	Bottom width, in feet.	Depth of water, in feet.	Area, in square feet.	Hydraulic radius.	Velocity, in feet per second.	Theoretical discharge, in second-feet.	Kutter's coefficient.
1	282.0	0.00008	20	7.5	206.25	5.01	1.59	326.0	0.025
2	743.6	0.00016	45	7.0	364.0	5.62	2.35	825.0	0.025
3	1 100.0	0.00015	70	7.0	514.0	6.09	2.47	1 270.0	0.025

The first point is about 18 miles below the head of the ditch; the second is 20 miles below the first; and the third is 7 miles below the second. The head of the ditch was encompassed, and no inflow was found; in fact, little water was discovered until a point about 10 miles below the head of the ditch was reached.

The approximate areas above each point are as follows: (1), 140 sq. miles; (2), 300 sq. miles; and (3), 330 sq. miles.

On September 3d, 1913, the following results were obtained:

at (1) 94.7 sec-ft. = 0.68 sec-ft. per sq. mile.
at (2) 111.7 " = 0.37 " " " "
at (3) 84.1 " = 0.26 " " " "

It was assumed that some error in the observation might have been made, and on September 23d, 1913, the following results were obtained:

at (1) 97.8 sec-ft. = 0.70 sec-ft. per sq. mile.
at (2) 120.8 " = 0.40 " " " "
at (3) 90.8 " = 0.28 " " " "

Table 9 gives the monthly rainfall records at New Madrid and at Cairo, stations at opposite corners of the District, from April, 1913, to September, 1913, inclusive.

TABLE 9.—Rainfall at New Madrid and Cairo.

Station.	April.		May.		June.		July.		August.		Sept.	
	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.
New Madrid....	3.96	3.88	1.58	4.90	1.27	4.40	2.73	4.38	1.09	3.79	5.41	2.95
Cairo.....	2.27	3.57	0.87	3.83	1.07	4.31	4.19	3.45	1.54	2.93	2.22	2.47

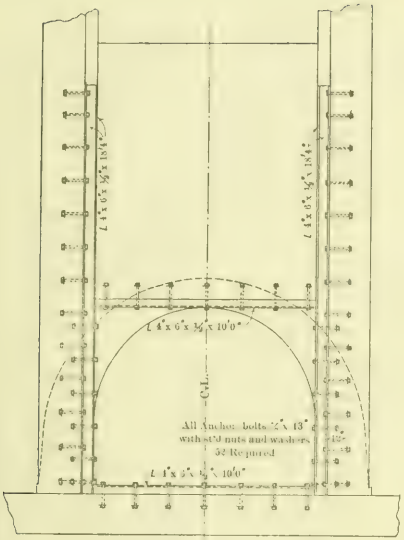
It is generally believed that the sand stratum beneath the high lands becomes saturated with water when the river is at high stages, and the water gradually drains out during the summer. The drop between the results obtained at Points 2 and 3 has not yet been explained satisfactorily.

Drainage of Ridge Lands and New Madrid.—The construction of the high-level ditch along the Sikeston Ridge and the river levee at New Madrid obstructs the drainage, and a system of collecting ditches and inverted siphons has been planned for the relief of these points.

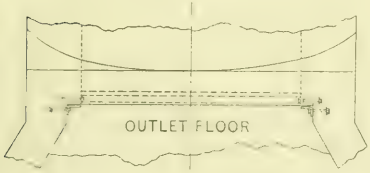
Pumping Plants.—It is physically possible to carry the water collected by the low-level ditch through the Sikeston Ridge to an outlet through the St. Francis River and thus eliminate all pumping costs, but, under the present legal conditions, this outlet cannot be effected.

It is necessary, therefore, to provide adequate pumping facilities before the reclamation of the low lands is complete; owing to the undeveloped state of these lands, however, it is not advisable to put the cost thereof on such lands at present. The area of this pumping territory is approximately 140 000 acres. The run-off of approximately 46 000 acres can be accommodated in storage.

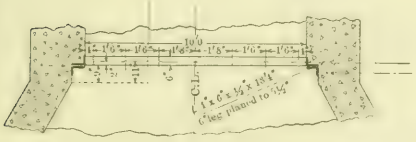
Storage Basins.—The Ten Mile Pond Basin includes what is known as Ten Mile Pond, Four Mile Pond, Six Mile Pond, Eagle Pond, and various smaller ponds and lakes. These will operate as one basin because James Bayou will act as a connecting or equalizing channel throughout this territory.

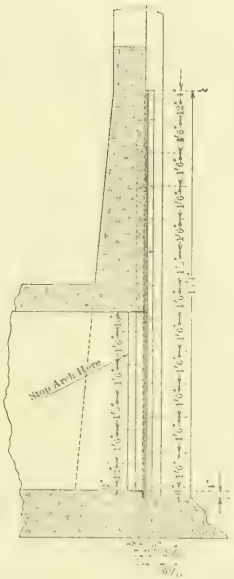


DETAIL OF GUIDE-ANGLES AT OUTLET



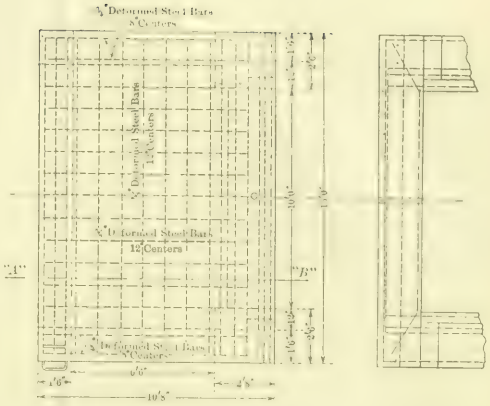
OUTLET FLOOR



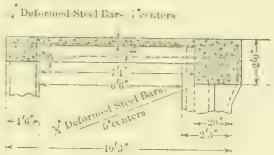


SECTION ON CENTER LINE

Note.—All exposed corners to be beveled or rounded. Platform floor to be finished with a ½" coat of 1:2 cement mortar paved at time of concreting.

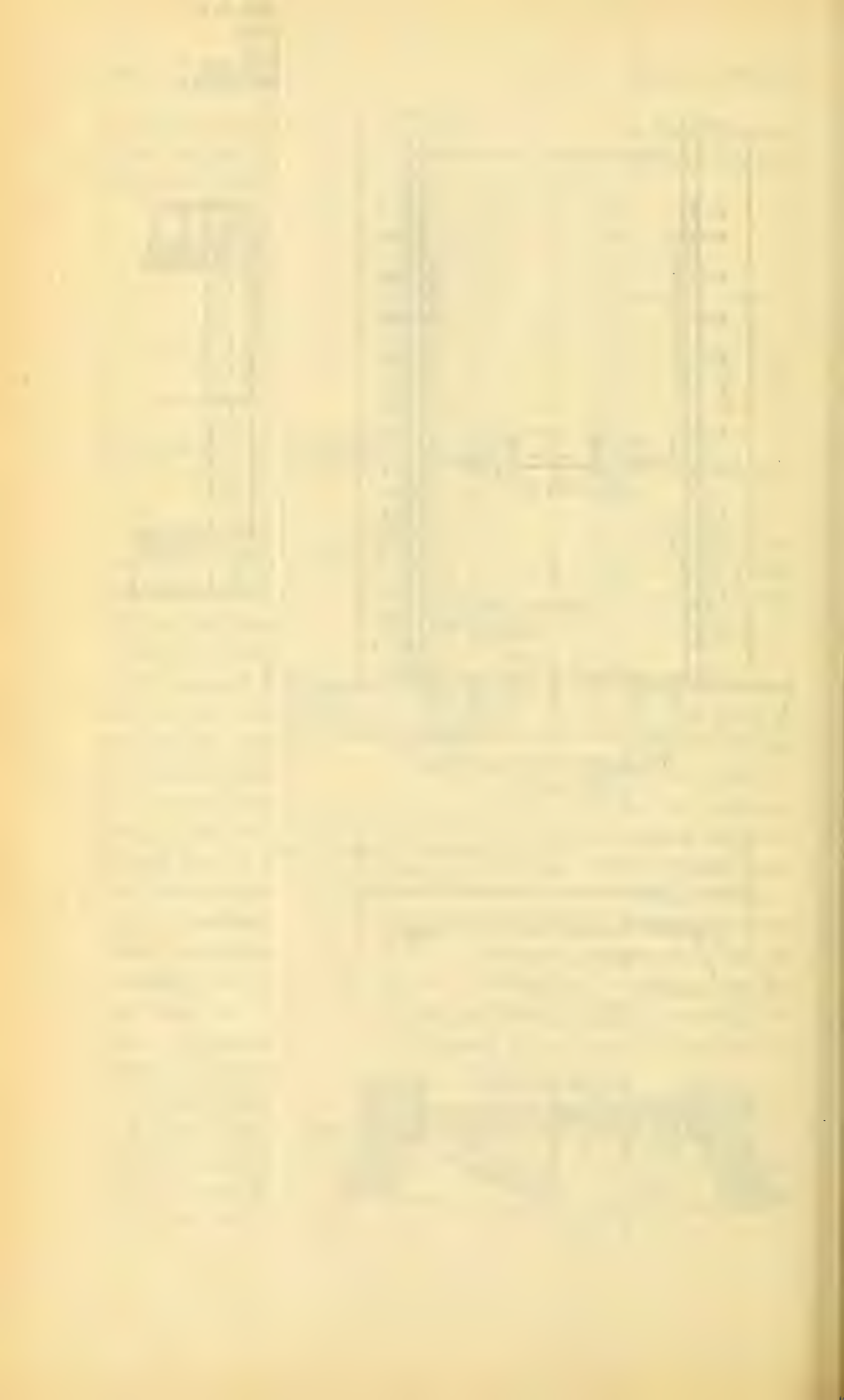


PLATFORM REINFORCING



SECTION *A-B*

DETAILS
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.



A storage level at Elevation 299.0 has been decided on, because that will give the greatest storage with the least damage to adjoining lands. The total area covered by this storage basin will approximate 6 820 acres, all of which, except 1 300 acres, will have no water except immediately after hard rains and at times when the flood-gates in St. John Bayou have been closed for a long period.

The storage in this basin will amount to approximately 12 250 acre-ft. The discharge capacity of the low-level ditch is somewhat greater than 1 500 acre-ft. per day; therefore, the impounded water would be discharged in approximately 6 days. To accomplish this, two spillway dams with flood-gates will be required. At ordinary times these gates will be left open to pass the natural drainage. When the gates in St. John Bayou are closed, the spillway dam gates will be closed. If the water rises above the crest of the spillway, which will be placed at an elevation of 299.0 ft., it will overflow and pass on down the low-level ditch and be held behind the gates in St. John Bayou.

One dam is to be placed in the Lee-Rowe Ditch near the point where it leaves Ten Mile Pond, and the other near the head of the low-level ditch. No land between the two dams is at an elevation lower than about 303.0 ft.; the land south of the dam in the low-level ditch is still higher, and no water can pass from the basin to a lower level except over the spillways.

Eagle Nest Cypress Basin is near the center of Township 23 North, Range 15 East, and is bounded on the north and west by a high bank, averaging about 14 ft. above the bottom of the basin. The ground rises gently from the bottom of the basin in a southeasterly direction. The bottom of this basin is at an elevation of about 286.0 ft., and the ground at the Mississippi River is at an elevation of 300.0 ft.

This basin is separated from the flat lands along the St. John Ditch and St. John Bayou by a ridge varying in elevation from 296.0 to 298.0 ft. The East Bayou cuts through this ridge, connecting it with St. John Bayou.

The flats along St. John Ditch are at an elevation of from 292.0 to 295.0 ft., therefore, without some kind of a dam in East Bayou, a storage level at an elevation of 292.0 ft. could not be reached without encroaching on the lands along St. John Ditch and St. John Bayou.

With a dam across East Bayou, a storage level of 296.0 ft. could

be maintained, but practically all the lands from the ridge to the river would be overflowed.

If a storage level is placed at an elevation higher than 292.0 ft., it is doubtful whether the cost of flood-gates and dam and reservoir would be warranted by the storage obtainable, hence no plan is proposed to use this basin for storage.

Storage Observations.—Observations to determine the quantity of rainfall that will reach the storage basins have been made for the Big Lake Drainage and Levee District of Missouri, which adjoins the St. John Levee and Drainage District of Missouri on the north, by L. T. Berthe, Assoc. M. Am. Soc. C. E., Engineer for that district. The results of these observations show that only about 21% of the rainfall finally reaches the basin.

The records show that the flood-gates of St. John Bayou would be closed on an average about 90 days each year, also that the average rainfall for this period has been 0.123 in. per day. Assuming that 25% of the rainfall will reach the basins, there will be 10 598 acre-ft. in storage, and under these conditions the run-off from 46 000 acres will be cared for.

These conditions, of course, are taken as an average, but a variation either way may be expected.

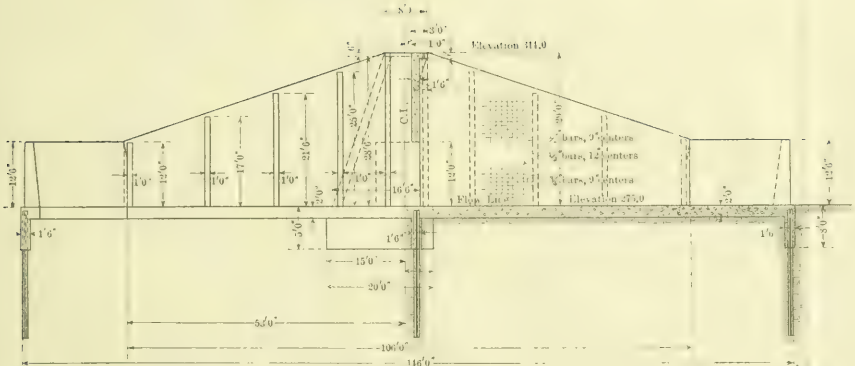
Flood Frequency.—All work for the district has been based on the flood heights as recorded for the New Madrid gauge. Table 10 shows the number of days each year that the river was above the different stages, and hydrographs for the New Madrid and Cairo gauges have been plotted.

An interesting comparison of the gauges is shown by the hydrographs, and the variation of the readings of the two can be compared for different stages of the river. Zero of the New Madrid gauge is at an elevation of 262.59 ft., Memphis datum, and that of the Cairo gauge at an elevation of 277.71 ft.

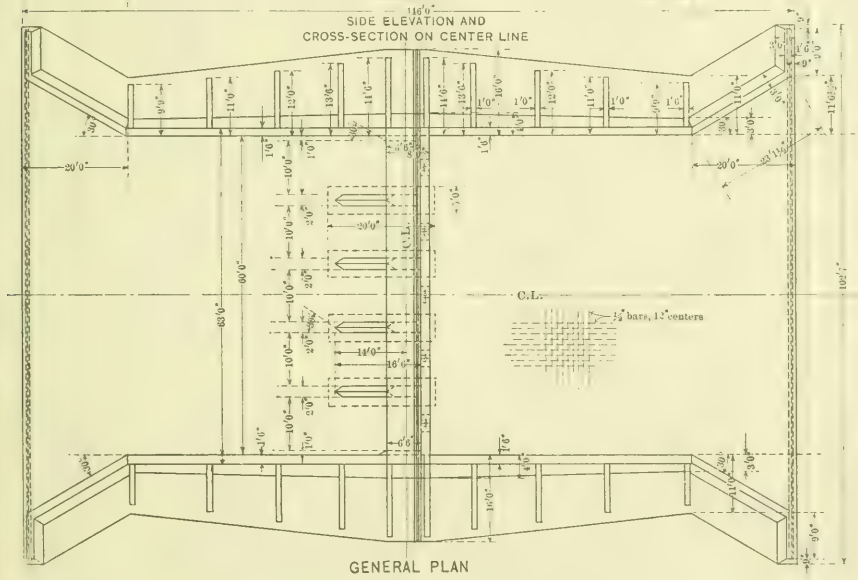
Table 11 shows the number of years that each stage has been reached out of the total number of years that the records have been kept, and gives the best possible conclusion as to the frequency of the various stages.

Estimates.—Yardage estimates for the levee construction show 6 197 500 cu. yd. of embankment. This work is variously estimated at

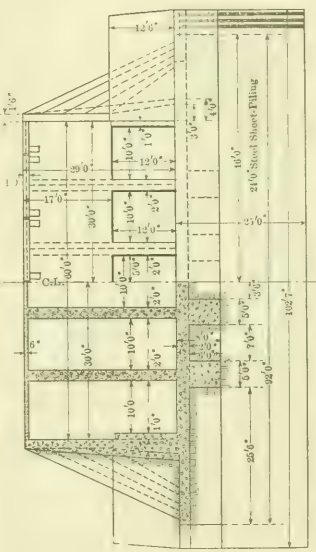
ST.JOHN BAYOU OUTLET
SLUICEWAY
ST.JOHN LEVEE AND DRAINAGE DISTRICT
OF MISSOURI



SIDE ELEVATION AND
CROSS-SECTION ON CENTER LINE



GENERAL PLAN



END ELEVATION AND
CROSS-SECTION ON CENTER LINE

from 12 to 25 cents per cu. yd., depending on the class of work and the quantity of grubbing.

TABLE 10.—River Stages at New Madrid.

Year.	Number of Days Above River Stage of:							
	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.	42 ft.	44 ft.	46 ft.
1880	No records at New Madrid.							
1881	" " " " "							
1882	" " " " "							
1883	" " " " "							
1884	56	46	36	16	11	0	..	..
1885	0	..	..	..	..	..	..	..
1886	27	24	20	15	5	0	..	..
1887	43	38	24	0	..	..	..	..
1888	22	12	1	0	..	..	..	..
1889	0	..	..	..	..	..	..	..
1890	64	52	37	3	0	..	..	..
1891	67	53	8	0	..	..	..	..
1892	No record at New Madrid.							
1893	61	33	16	6	0	..	..	..
1894	0	..	..	..	..	..	..	..
1895	0	..	..	..	..	..	..	..
1896	0	..	..	..	..	..	..	..
1897	59	54	46	33	8	0	..	..
1898	38	28	15	6	0	..	..	..
1899	43	34	25	0	..	..	..	..
1900	1	0	..	..	..	..	..	..
1901	13	4	0	..	..	..	..	..
1902	8	0	..	..	..	..	..	..
1903	78	61	18	12	0	..	..	..
1904	30	20	14	5	0	..	..	..
1905	20	13	0	..	..	..	..	..
1906	22	18	10	0	..	..	..	..
1907	51	32	16	10	0	..	..	..
1908	83	50	16	0	..	..	..	..
1909	66	40	23	12	0	..	..	..
1910	14	4	0	..	..	..	..	..
1911	16	12	5	0	..	..	..	..
1912	71	57	47	36	23	16	2	0
1913	62	55	49	39	20	15	8	0
1914	9	0	..	..	..	..	..	..

TABLE 11.—Flood Frequency at New Madrid, Mo.

Gauge reading.	Number of years stage was reached in 29 years.	Corresponding elevation.
32	25	294.59
34	22	296.59
36	19	298.59
38	12	300.59
40	5	302.59
42	2	304.59
44*	2	306.59

*The highest recorded reading on the New Madrid gauge is 44.6 ft., corresponding to an elevation of 307.19. This stage was reached on April 9th, 1913.

The total excavation for the various ditches is placed at 10 715 000 cu. yd., and this work is estimated at from 8½ to 18 cents per cu. yd. Clearing of right of way is estimated separately.

The total cost of the improvement, as planned, without the pumping plant, but including all dams, flood-gates, sluice-ways, siphons, etc., is $3 750 000.

Order of Construction.—On account of the cost of the system, it is certain that the construction of the high-level ditch will be delayed until some future time. The work decided on at present consists of the extension of the levee about 17 miles down the river from Dorena, the cutting of the low-level ditch, and the construction of a levee along the river front at New Madrid and along the east edge of the Sikeston Ridge to high land above overflow, in order to prevent the passage of water across the Sikeston Ridge at high stages of the river.

The work outlined will fit in with the complete plan, with the exception of that part of the Sikeston Ridge Levee which extends along the ridge beyond the location of the high-level ditch.

It is necessary to investigate the probable effect of this procedure, as no increase in the size of the low-level ditch is contemplated, although its contributing area is greatly increased.

Approximately, 172 600 acres will now be tributary to the Ten Mile Pond Basin, and, at $\frac{7}{32}$ in. run-off in 24 hours, the inflow to the basin will be 1 364 sec-ft.

The estimated discharge of the low-level ditch, with water running 7 ft. deep, is 825 sec-ft., but where the ditch leaves the Lee-Rowe Ditch a depth of 14 ft. can be reached without flooding the banks.

The water will back up in the Lee-Rowe Ditch and the Ten Mile Pond Basin until it attains a depth in the low-level ditch sufficient to produce a discharge equivalent to the volume of water entering the basin, after which no additional rise will occur. A depth of 9.5 ft. in the ditch will produce this discharge, and will produce a water level at an elevation of about 296.0 ft. in the ditch; the level in the basin will then probably reach 298.0 ft.

It is believed that, with the low-level ditch alone in operation, under the usual conditions, little or no damage will result, except in cases of excessive rainfall.

A maximum rainfall of 9.6 in. in 48 hours has been recorded for these localities. Assuming that 25% run-off results, this will produce 0.2 ft. in 48 hours, or 8 700 sec-ft. With a depth of flow of 13.5 ft. in the low-level ditch, a discharge of 2 730 sec-ft. results, and the excess,

of about 6 000 sec-ft., must be held in storage until the low-level ditch has again emptied the basin, which will take about 7 days.

A depth of flow of 13.5 ft. in the low-level ditch is attained with a water level of about 300 ft., and some flooding of the lands above 300 ft. will occur.

The low-level ditch in Eagle Nest Cypress Basin has an effective depth of only 5 or 6 ft., and the ditch discharge here is 655 sec-ft. With the usual conditions of run-off ($\frac{7}{32}$ in. in 24 hours), the ditch brings to the basin 1 364 sec-ft., so it is seen that some flooding of the lands along the banks will occur, but, under the condition that the ditch brings 2 730 sec-ft. to the basin, a ponding will occur until the ditch leading from the basin attains an equivalent discharge, which will be reached with a water level at 293.0 ft. elevation and a depth of 14 ft. in the ditch. Therefore, under the maximum conditions, all lands in this basin below an elevation of 293.0 ft. will be flooded.

The frequency of these excessive storms must also be considered. The records for Missouri show that a maximum hourly rainfall of 4.74 in. has been observed. The maximum 1-day storm recorded shows a rainfall of 8.0 in., and 16 storms with a rainfall of 5 in. or more in 24 hours have been observed. The maximum 2-day storm recorded shows a rainfall of 9.6 in., and 17 storms with a rainfall of 5 in. or more in 48 hours have been recorded.

The records of the Cairo Station, kept since 1872, show thirty-five 1-day storms with a rainfall of 2 in. or more; fourteen storms with 3 in. or more; six storms with 4 in. or more; and two storms with 5 in. or more.

Assessments of Benefits.—The Missouri laws provide that the cost of construction must be apportioned over the lands or other property receiving benefits through the construction of a plan for reclamation, in the proportion of the benefits received. A tax equal to the amount of benefits can be collected, but where the benefits received only equal the cost of construction, it is a useless expenditure of money, and the Court will not recognize the project.

Three commissioners are appointed by the Court, under which the District is organized, on the filing of the plan for reclamation. It is their duty to go on the ground to determine the value of all property or lands to be acquired for right of way or other works of the District, as set out in the plan for reclamation, either within or without the Dis-

trict. They shall assess the amount of benefits and damages, if any, that will accrue to lands or other property from carrying out and putting into effect the plan for reclamation.

The chief engineer of the District or one of his assistants shall accompany the Commissioners at all times when engaged in their duties, and shall render his opinion in writing when called for.

The following outline, submitted by the writer, for the assessment of benefits in this District was adopted by the Commissioners, and will show the part taken by the engineering features of a plan for reclamation in the assessment of benefits:

The plan adopted for this District does not provide complete reclamation for all the lands within the District, but the proposed work when completed will form a part of a general plan providing complete reclamation. The construction of the Sikeston Ridge Levee will protect the lands on the ridge and in the City of New Madrid from overflow, and the benefits derived by the ridge lands will be those resulting from protecting them from the scour to which they are subjected by the severe current passing over them at high-river stages. The damage that this current has caused, by washing away the rich soil, cutting deep holes and channels across the lands, washing out roads and railroads, tearing down fences, and moving buildings from their foundations, etc., property damage in the City of New Madrid, and loss of merchandise, can be estimated at nearly $300 000 for 1913 alone.

The ridge lands and the city can be considered separately from the low lands, as the proposed work is not connected.

Benefits to New Madrid.—The different parts of the town are practically at the same elevation, and will be overflowed at the same stage of the river, so that all property may be considered equal in this respect. The damage done by the water, then, will depend on the value of the property. No personal property can be assessed.

It will be necessary to make a complete valuation of all real property within the city limits, placing a fair value on each lot or tract and adding thereto the value of buildings thereon. A representative property can then be selected and the flood damage to it estimated. This damage then will be the total which the property will sustain during a period determined from past overflows, which is about 7 years. The damage divided by 7 and this amount capitalized at 6% will be the amount of the benefit. A percentage is then found between

the benefit and the value of the property, and this percentage of the value of any property is taken as the benefit to that property.

Ridge Lands.—The benefits to ridge lands are determined in part by the elevation of the respective tracts. Lands on the east side of the ridge being somewhat lower, are subject to overflow before those on the summit and west side. The lands at a higher elevation are subject to the severe cutting of the current, as are those on the west side of the ridge. The benefit to a tract of land, therefore, will depend largely on the elevation and location and the probability of overflow, and should vary from $5 to $30 per acre.

Bottom Lands.—The overflow lands of the District are for the greater part not under cultivation, but are timber or cut-over lands. The value of those which are under cultivation will range from $35 to $45 per acre, and that of the remainder varies from $5 to $15, depending on the quantity of timber thereon.

As the plan for reclamation for this District proposes to extend the levee down the river from Dorena about 17 miles, there is still a gap, between the end of the levee and the levee at New Madrid, of about 6 miles. This leaves some of the lands of the District still subject to overflow from back-water from the Mississippi River.

It is not possible to show a benefit to the lands above back-water overflow, therefore, although there is a large area in the district above overflow, it cannot be assessed for benefits under this plan.

As the lands assessed are subject to overflow from back-water, it will be necessary to consider the effect of the extension of the levee on the water level over these lands at flood stages of the river.

During recent overflows, no true back-water elevations were obtained, as there were several breaks in the levees above Dorena, and the water from the Mississippi above Dorena raised the level of the water over these lands.

What the result of the 1913 flood would have been had the levees held is a matter of conjecture. The water level at Dorena was established at approximately 315.6 ft., but, had there been no breaks in the levees above, it would have undoubtedly reached an elevation of 316.0 ft., and this will be taken as the possible elevation of the back-water on the lands of the District.

Just what difference would be found between this level and the level of the water at the extremity of the overflow is not known, but it is

certain that the elevation at the limit of overflow will not be quite as high as that at its origin. From the information at hand, it may be safely stated that this difference will be about 1 ft. in 9 miles, depending, of course, on the duration of the flood. The shorter the flood period, the greater the difference will be found, and if the flood is of sufficient duration, the water may seek nearly a level surface.

With the extension of the levee as proposed, the origin of the back-water will be 17 miles farther down stream. Considering the fall of the river and the increased flood height with the waters confined, the elevation of the back-water at the origin will be reduced from 316.0 ft. to approximately 310.0 ft., and the same differences will result on the level of the back-water as before.

The 1913 flood is considered to have been one of the maximum floods in the Mississippi Valley. Floods which average nearly as high as that of 1913 are by no means rare, as those of 1882, 1883, 1884, 1897, 1903, 1907, and 1912 were far above the average years.

The construction of the levees at Cairo and along the Mississippi River has also increased the flood height of the more recent overflows, so, under the same conditions, the earlier floods might have reached the same stage as that of 1913.

The frequency of floods must be considered, as lands subject to overflow every year would receive far greater benefit than those subject to overflow but once in 10 years, were the overflows taken away completely.

Therefore, it must be seen that the benefits which a tract of land receives from this improvement depends: (1), on the elevation of the tract; (2), on the frequency of overflow; and (3), on the improvements on the tract.

By comparing the stages of the river at New Madrid with the elevation of the flood-plane at the end of the proposed levee, and taking the frequency of these stages, the probable stage of overflow of lands at various elevations may be determined and the period between overflows taken from the flood frequency.

By comparing the elevation of the back-water from Dorena with that from the end of the proposed levee, and combining with the frequency, we have the following scale of assessment:

Lands with elevation of from 294.0 to 296.0, 5% of maximum benefit.

Lands with elevation of from 296.0 to 297.0, 10% of maximum benefit.

Lands with elevation of from 297.0 to 298.0, 20% of maximum benefit.

Lands with elevation of from 298.0 to 299.6, 30% of maximum benefit.

Lands with elevation of from 299.6 to 300.6, 40% of maximum benefit.

Lands with elevation of from 300.6 to 302.6, 50% of maximum benefit.

Lands with elevation of from 302.6 to 304.6, 75% of maximum benefit.

Lands with elevation of from 304.6 to (306.6-309.0), 90% of maximum benefit.

Lands with elevation of from (306.6-309.0) to 310.0, 50% of maximum benefit.

Lands with elevation of from 310.0 to 312.0, 30% of maximum benefit.

Lands with elevation of from 312.0 to 314.0, 10% of maximum benefit.

Lands with elevation of from 314.0 to 315.0, 5% of maximum benefit.

TABLE 12.—ELEVATION—FLOOD-FREQUENCY TABLE.

New Madrid gauge reading.	Number of years that this stage was reached in 29 years past.	Corresponding elevation at New Madrid.	Corresponding elevation at end of proposed levee.	Corresponding elevation at Dorena, inside of levee.	Corresponding elevation at Grassy Pond Cypress.	Corresponding elevation at Selkirk.	Present water level at Dorena.
32	25	294.59	296.5	295.0	294.5	295.5	299.6
34	22	296.59	298.5	297.0	296.5	297.5	302.1
36	19	298.59	300.6	299.1	298.6	299.6	304.6
38	12	300.59	303.0	301.5	301.0	302.0	307.1
40	5	302.59	305.8	304.3	303.8	304.8	309.6
42	2	304.59	308.0	306.5	306.0	307.0	312.4
44	2	306.59	310.0	308.5	308.0	309.0	315.1

An interesting point in this scale is the peak in the benefits. The value per acre of absolute protection from overflow was taken at $24, and the different rates derived.

Lands in Levee District No. 1, Mississippi County, having borne the cost of construction of the levees along the Mississippi River, which are now to become a part of the system and lessen the cost of protection to lands outside of Levee District No. 1, must be allowed credit in reduced benefits for the work constructed by them. This credit is to be worked out as follows: Determine the cost of these levees and the benefit derived by Levee District No. 1 lands. Determine the proportion of the benefits of the proposed improvements between the lands in Levee District No. 1 and all others; then this proportional part of the benefits due to the existing levee will give the reduction due to lands in Levee District No. 1. As the plan includes the cutting of the low-level ditch, an equitable adjustment of its cost must be made.

It would appear that, before the opening at James Bayou and the Lee-Rowe Ditch can be closed and any benefits derived from the construction of the levee, this ditch is necessary. Little benefit is derived from the use of this ditch for local drainage by any lands, and such benefits should be distributed over the lands getting levee benefits and in the same proportion as the latter. This ditch cannot be considered as a drainage project, as it is made necessary absolutely by the construction of the levees and, therefore, is properly chargeable as a levee project.

A part of the Lee-Rowe Ditch is used under this plan for the low-level ditch, and the cost of the section used was about $8 000. It may be considered that, as the cost of closing this ditch was in excess of the cost of the section, and as the cutting of this ditch directly into the river was a damage to a large part of the lands in the district, the lands responsible for the cutting of the Lee-Rowe Ditch are not entitled to any credit for the part used.

Railroads and Other Property.—The Missouri law provides that railroads shall be assessed according to the increased physical efficiency and decreased maintenance cost by reason of the protection to be derived from the proposed works and improvements.

Two railroads, the St. Louis and San Francisco and the St. Louis Southwestern, suffered considerable loss due to flood damage and loss of traffic during the past two high waters. Several washouts occurred which they found necessary to span by pile trestles in order to get their tracks open in the least possible time after the flood passed.

These trestles are no longer necessary, and the benefit derived from closing them was worked out in a tabular form, as follows: Column 1, showing the length of trestle filled; Column 2, the yardage in the fill; Column 3, the cost of embankment; Column 4, maintenance cost of trestle over embankment at $1 per ft. per year; Column 5, maintenance cost capitalized at 5%; Column 6, net benefit by eliminating trestle, the difference of cost in Columns 3 and 5.

The benefits derived by the railroads through the elimination of flood damage and loss of traffic can be estimated only roughly.

No records of the cost of track renewals and repairs are available to the District. These records should be produced in Court in order that a correct basis may be established for the assessment. The loss of traffic cannot be fixed definitely, as can the flood damage, but it can be estimated by those who know the conditions of train operation, such as freight and passengers handled.

The total of flood damage and loss of traffic, distributed over the period of years that this damage may be expected to occur, and capitalized at 5%, will be the benefit assessed against the railroad roadway and other property.

Telegraph and telephone lines operating within the overflow are subject to damage to toll lines, by the breaking of the lines and loss of tolls, due to the population leaving the belt of danger, and some assessment of benefits should be placed on those properties.

The law provides for the assessment of public roads and, in some cases, this is done. In this case there is little benefit to be derived, as most of the roads are still subject to overflow and are not improved, so that it is not advisable to assess them.

The plan of the high-level ditch and the organization of the three counties into one district for increased efficiency in maintenance of protection works was originated by L. T. Berthe, Assoc. M. Am. Soc. C. E., who has aided the work for this district as far as possible.

J. A. Ockerson, Past-President, Am. Soc. C. E., Member of the Mississippi River Commission, was retained as Consulting Engineer.

The writer has had charge of this work for Charles H. Miller, M. Am. Soc. C. E., of the Miller Engineering Company, of Little Rock, Ark., to whom he is indebted for assistance and advice.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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COMPUTING RUN-OFF FROM RAINFALL AND OTHER PHYSICAL DATA

By Adolph F. Meyer, M. Am. Soc. C. E.

To be Presented April 21st, 1915.

Synopsis.

The paper points out the necessity of basing conclusions with respect to stream flow on extended physical data. As most stream-flow observations available for the engineer's use extend over a comparatively few years, there is need for a method of computing run-off from other physical data for the purpose of extending and supplementing short-term stream-flow records.

Inasmuch as run-off or stream flow consists of the residual rainfall after all losses have been deducted, the paper treats first of the method for computing these losses.

The loss of rainfall through evaporation from water surfaces and from snow and ice is first discussed. The factors modifying the rate of evaporation and their relative importance, are considered, and curves are presented, based on observations and checked by an evaporation formula, for the computation of these losses for any given water-shed, from temperature records, and the area and character of the bodies of water found on the water-shed.

Evaporation from land areas is next discussed, with respect to its variation with temperature, season, rainfall, vegetal cover, topography,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

soil, and subsoil, and curves are presented for the determination of the loss of rainfall through evaporation from land areas for various temperatures and rates of rainfall. The values taken from these curves are to be modified by the use of certain coefficients based on water-shed characteristics.

The losses out of rainfall resulting from the transpiration of plants, as determined by various investigators, are briefly mentioned, and some of the underlying principles governing the transpiration of plants are pointed out. The water requirements of plants are discussed, and a curve is presented to aid in computing monthly transpiration losses.

A summary statement of the author's method of computing run-off is next made, and the method applied to fifteen widely different water-sheds: A brief statement of the outstanding physical characteristics of these water-sheds is given, to aid the reader in understanding the choice of coefficients used in the computations.

The variation of evaporation and transpiration losses for various types of rainfall distribution exemplified by the given water-sheds is discussed, and the necessity of taking monthly temperatures into consideration in every method of computing evaporation and transpiration losses is pointed out.

A summary is given of the physical data and computations for the fifteen water-sheds discussed, together with detailed computations for characteristic years. The water-sheds used range in area from 100 to 37 000 sq. miles, with a variation in average annual rainfall of from 14.8 to 61.9 in.; a variation in mean annual temperature of from 37 to 66°, and a variation in mean annual run-off of from less than 1 in. to more than 50 in. Reasonably close agreement between computed and observed annual run-off under these widely varying physical conditions is shown in every instance.

The monthly distribution of run-off is discussed, curves are presented, and the author's method is applied in detail to the Root River water-shed, in Minnesota.

To show the variation in annual rainfall and run-off, and to emphasize further the need for long-term records, two stations are selected and curves presented, giving 5-year progressive mean values and cumulative mean values of rainfall and run-off. On one of the

water-sheds selected, the 5-year mean run-off, for example, varies from 66 to 142% of the 17-year mean run-off.

Introductory Comments.

From time to time various curves and formulas designed to give the annual yield of water from any given water-shed, and its distribution throughout the year, have been presented. Perhaps the most common expression of these quantities has been in terms of percentage of precipitation. Whenever this method has been adopted, great variations in run-off for the same quantities of precipitation have been noted. In fact, the lack of direct relationship between rainfall and run-off is a fact of common observation among those who have made a study of such data. Run-off, for a given month, considerably in excess of the rainfall for the same month, is not an exceptional occurrence on many streams of the country. For the same annual rainfall the annual run-off occasionally varies by nearly 100% on the same stream.

These facts are well illustrated by Fig. 1.* The run-off for April, in Fig. 1, shows a variation of from 5 to 200% of the rainfall on the same water-shed. Moreover, the high percentage is for the lower rate of rainfall. The run-off for September shows a variation of from 2 to 140% of the rainfall for practically the same precipitation on the same water-shed. The annual rainfall for one of the streams on Fig. 1 varies from 6.7 to 11.97 in., or about 80% for practically the same annual rainfall.

In attempting to express the relationship between rainfall and run-off, Vermeule† used a constant plus a percentage for the various months of the year, and varied this relationship on different water-sheds with the mean annual temperature.

Justin‡ expressed annual run-off by an equation consisting of a coefficient (which varied with slope and mean annual temperature for different water-sheds) multiplied by the square of the annual rainfall.

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* Compiled from "The Flow of Streams and the Factors That Modify It," by D. W. Mead, M. Am. Soc. C. E.

† Water Supply of New Jersey, 1894; and Annual Report, State Geologist of New Jersey, 1899.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

Babb* used curves giving the monthly run-off to be expected from any given water-shed in the various parts of the country, in terms of a percentage of the total annual run-off. The latter was computed from the annual rainfall by using a percentage relationship derived from a study of the mean observed relation between rainfall and run-off on a number of streams in various parts of the country.

Rafter† used curves, which, in general, are graphs of an exponential equation, for the purpose of showing the relation between rainfall and run-off by "Storage", "Growing", and "Replenishing" periods.

Newell‡ expressed the general relationship between rainfall and run-off by two typical curves, one for streams in mountainous regions and the other for streams draining basins with broad valleys and gentle slopes.

Other writers have held that 20 in. of rainfall are required to supply evaporation and transpiration losses, and that practically all precipitation greater than 20 in. appears as run-off.

Although recognizing the able manner in which the subject of stream flow has been discussed in the technical press during past years, the writer cannot refrain from expressing the belief that the relations between rainfall and run-off indicated by the curves and formulas just referred to, are, in a varying degree, generalizations, which bring out class likenesses, but obscure the individual characteristics of run-off from different water-sheds, resulting from differences in the character and distribution of the rainfall, and the effect of temperature, vegetal cover, topography, soil, and subsoil on the disposal of rainfall.

The writer regrets that he cannot speak from the experience of a lifetime devoted to hydrological investigations, yet during the past 10 years he has been almost continually connected with river work of various kinds, and the following conclusions are the result of some little study and observation. These conclusions are here stated with full realization of their limitations, and the probability of some future modification, yet with sufficient confidence in their usefulness as applied, particularly to supplementing stream-flow data, to warrant their publication. Let it be understood at the outset, that the writer does

* *Transactions*, Am. Soc. C. E., Vol. XXVIII, p. 323.

† Water Supply and Irrigation Paper No. 80, U. S. Geol. Survey.

‡ Fourteenth Annual Report, Part 2, 1892–1893, U. S. Geol. Survey.

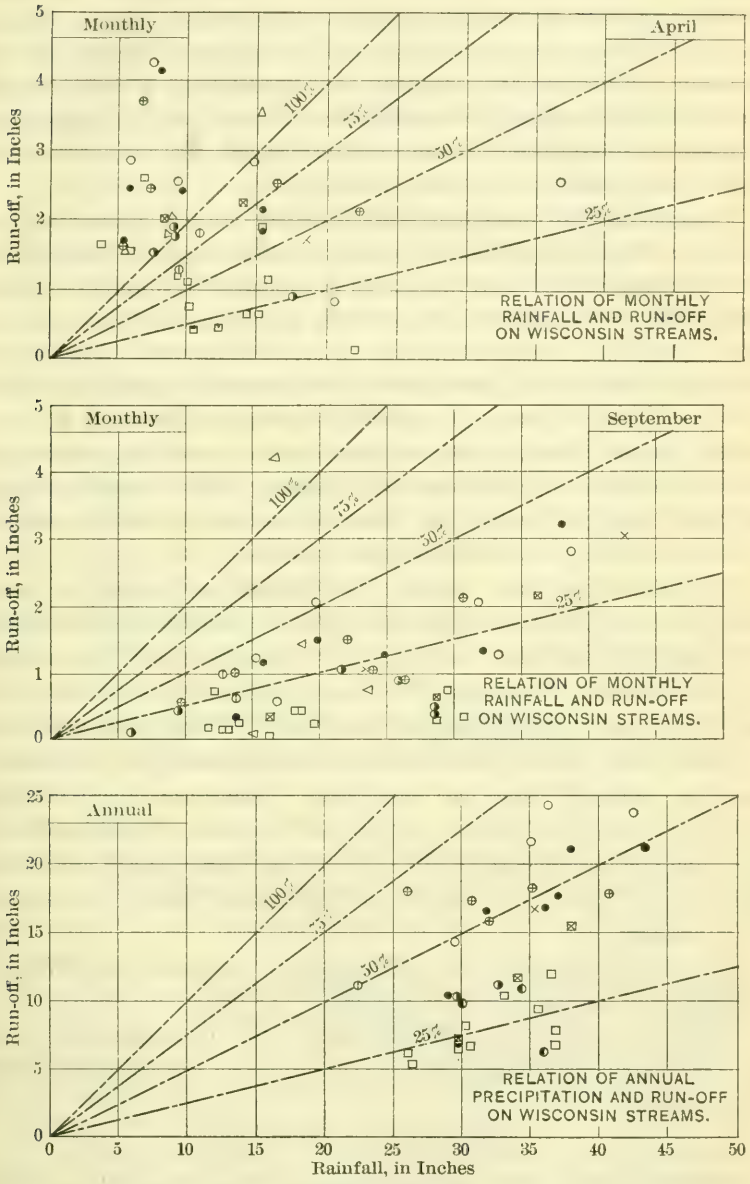

Monthly
April
100%
75%
50%
25%
RELATION OF MONTHLY RAINFALL AND RUN-OFF ON WISCONSIN STREAMS.
Run-off, in Inches
Monthly
September
100%
75%
50%
25%
RELATION OF MONTHLY RAINFALL AND RUN-OFF ON WISCONSIN STREAMS.
Run-off, in Inches
Annual
100%
75%
50%
25%
RELATION OF ANNUAL PRECIPITATION AND RUN-OFF ON WISCONSIN STREAMS.
Run-off, in Inches
Rainfall, in Inches

FIG. 1.

not claim to have discovered a method of computing daily or even monthly run-off from rainfall and other physical data which obviates the necessity for stream measurements. He believes, however, that he has found a method of computing the annual run-off from widely different water-sheds with considerable accuracy, and of computing a reasonable distribution of such run-off through the various months of the year for most of such water-sheds.

Considering the number of streams in the United States the discharge of which is of industrial importance, the number of stations at which stream flow is being measured is comparatively small, and the periods for which records are available are relatively short. If it takes from 30 to 40 years to secure an accurate measure of the mean annual rainfall at any given place, it is reasonably certain that the true means and extremes of run-off are compassed between at least as wide limits. Precipitation and temperature are being observed in the United States at more than 5 000 stations. Stream measurements are being made by Federal and State authorities and private parties, together, at about one-fourth as many stations.

Notwithstanding the valuable work being performed by these organizations on all too meager appropriations, relatively few stream-flow data are available. For most streams, only short-term records have been secured, covering by no means the extremes of high and low flow, or giving a dependable mean flow. If such measurements of stream flow as are available can be supplemented by reasonably accurate computed values, so as to give a long-term record of fair reliability, and covering more nearly the extremes of high and low flow, some of the uncertainty often attending efforts toward industrial utilization of the flow of streams and protection against floods may be eliminated.

It is the writer's hope that his method of computing run-off may at least be of assistance toward this end.

Before presenting the method itself, and its application to a number of widely different water-sheds, some of the basic data will be discussed.

Definition of Terms.

Rainfall.—This term, as here used, includes "snowfall" reduced to its equivalent in rain by melting the snow or by dividing by ten,

according to the accepted practice of the United States Weather Bureau. Wherever precipitation occurs as light snow at temperatures well below the freezing point, this ratio gives values about 10 to 30% high. However, inasmuch as at most observation stations a comparatively small portion of the precipitation occurs under the foregoing conditions, the resulting discrepancy in the annual precipitation is undoubtedly of less importance than other discrepancies, such as those resulting from differences in the placing of the rain gauge with respect to altitude and exposure. It is believed that, on the whole, however, precipitation measured with standard gauges, in accordance with Government practice, gives a quantity reasonably close to the truth.

Even though there may be some discrepancy in records of rainfall, giving quantities continually too large or too small, nevertheless, the computed run-off based on these data and on a comparison with available run-off measurements may be more accurate than the precipitation records on which the computations are based, since the constant error in the rainfall measurements becomes a part of the coefficient used in computing losses out of rainfall.

Evaporation.—This term is used to denote both the process of vaporization and the quantity of water which is vaporized and diffused into the atmosphere from land and water surfaces. Other writers have frequently included in the term "evaporation" both transpiration and deep seepage, in other words, all losses out of rainfall except run-off.

Transpiration.—This term is used to denote the water which escapes as vapor from the stomata of leaves, and the process by which such loss of moisture takes place. "Hygroscopic water", that is, the water retained in the vegetable substance produced, is inconsequential in this discussion.

Deep Seepage.—This is the water which passes down so far through the subsoil into the underlying rocks as to be lost to the given water-shed; it is also neglected on the streams discussed, although the possibility of its constituting an appreciable factor on one or two of the water-sheds considered, may here be noted.

Total Loss.—This term is used to denote the sum of evaporation and transpiration; and, whenever a long period of years is considered, the quantity "Precipitation minus Total Loss" is virtually equivalent

to "Run-off". The monthly values of "Precipitation minus Total Loss", and to some extent also the annual values, however—depending on the size of the water-shed, and all the other factors which influence the distribution of run-off—must be corrected for changes in ground and surface storage to give the run-off for the given time interval.

Run-off.—This term is used to denote stream flow maintained by ground and lake storage, together with surface flow resulting from heavy rains or the melting of snow.

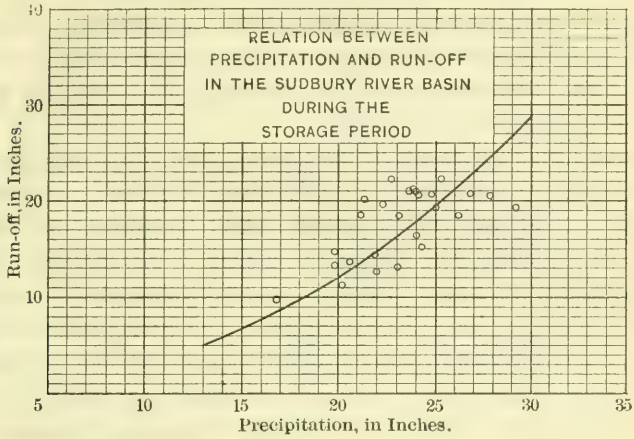
The "Water Year".

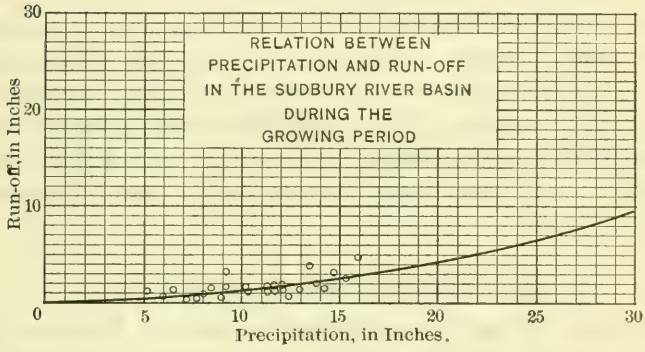
Frequent comment has been made on the fact that the calendar year is an inappropriate and conventional period into which to divide time, from a hydrological viewpoint. A period of 12 months, beginning December 1st and ending the following November 30th, has been used by many hydraulicians and called the "water year". This "water year" has again been divided into three periods, viz., December to May, inclusive, constituting the "storage" period; June to August, inclusive, constituting the "growing" period; and September to November, inclusive, constituting the "replenishing" period. Although this division of time is more logical than the calendar year, efforts to express run-off as a percentage of rainfall for each of these periods are considered by the writer hardly less futile than efforts to express run-off as a percentage of the monthly or annual rainfall.

Fig. 2* substantiates this view. The scale used in Rafter's diagrams for the growing and replenishing periods completely conceals the true lack of relationship between rainfall and run-off during these periods. At first glance one would conclude that the run-off during the growing and replenishing periods showed a much closer relationship to the rainfall than that of the storage period. On plotting the values for these two periods to a scale which results in a curve comparable to that of the storage period, however, quite the contrary is found to be the case, as shown in Fig. 3.

During the storage period, the run-off varies from 12.8 to 22.3 in., or practically 75% for a rainfall of between 22 and 23 in. During the growing period, on the same stream, the run-off varies from 0.72 to 3.07 in., or 325% for approximately the same rainfall. During

* Compiled from "The Relation of Rainfall to Run-off", by the late George W. Rafter, M. Am. Soc. C. E.


RELATION BETWEEN
PRECIPITATION AND RUN-OFF
IN THE SUDBURY RIVER BASIN
DURING THE
STORAGE PERIOD
Run-off, in Inches.
Precipitation, in Inches.


RELATION BETWEEN
PRECIPITATION AND RUN-OFF
IN THE SUDBURY RIVER BASIN
DURING THE
GROWING PERIOD
Run-off, in Inches
Precipitation, in Inches.

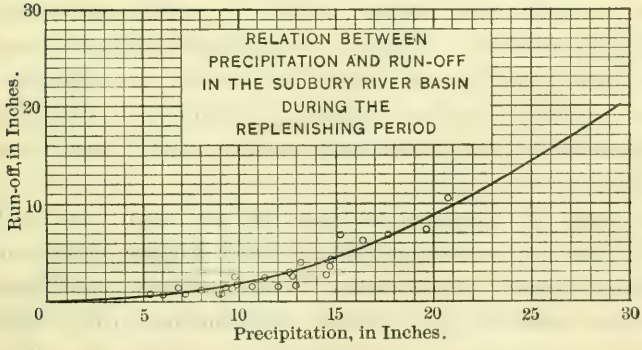

RELATION BETWEEN
PRECIPITATION AND RUN-OFF
IN THE SUDBURY RIVER BASIN
DURING THE
REPLENISHING PERIOD
Run-off, in Inches.
Precipitation, in Inches.

FIG. 2.

the replenishing period the run-off varies from 3.76 to 1.58 in., or 140% for rainfalls of 13.11 in. and 12.89 in., respectively. The entire annual run-off from this water-shed varied in 15 years from 12.69 in. for 39.70 in. of rainfall to 23.27 in. for 38.71 in. of rainfall, or practically 100% for the same rainfall.

The writer usually takes as his rainfall year, in northern latitudes, the 12-month period beginning November 1st, and as the corresponding run-off year the 12-month period beginning the following March 1st. Stream flow during the winter, in the northern half of Minnesota, for example, is almost entirely independent of the precipitation during these months, because such precipitation is practically all stored as snow. Stream flow, in such latitudes, is dependent on the ground-water stored during the previous open seasons.

In the greater portion of the United States, a 12-month period beginning August, September, or October 1st, when the ground and surface storage are both reduced to a minimum, affords a satisfactory "water year". Usually, however, the annual yield of a water-shed, even in such "water years", is modified somewhat by ground storage.

The writer computes the annual run-off entirely by calendar months, without any attempt to adhere to a division of the year into "storage", "growing", and "replenishing" periods, or into spring, summer, fall, and winter seasons.

Evaporation from Water, Snow, and Ice.

Although the laws of evaporation, as stated by Professor Thomas Tate, are thirteen in number, the writer has found that in computing stream flow only three factors tending to modify evaporation need usually be considered. These are, in the order of their importance: temperature, humidity, and wind. The last two are relatively unimportant within the ordinary range of variation, but some allowance from water-shed to water-shed, and month to month, must be made for wide differences.

The writer has accepted the principle, first enunciated by Dalton, that the rate of evaporation from a water surface, other conditions remaining constant, varies almost, if not quite, as the difference between the maximum vapor pressure corresponding to the temperature of the water, and the pressure of vapor present in the atmosphere.

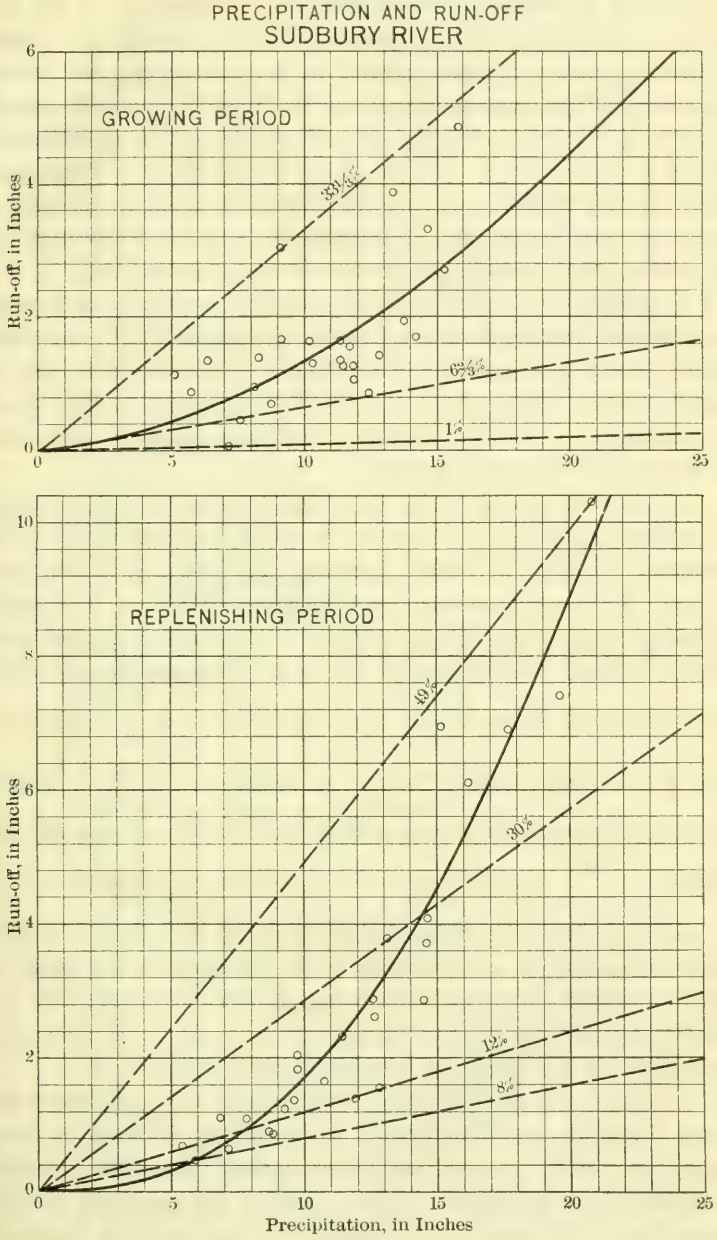

PRECIPITATION AND RUN-OFF
SUDBURY RIVER
GROWING PERIOD
33⅓%
6⅔%
1%
Run-off, in Inches
REPLENISHING PERIOD
49%
30%
12%
8%
Run-off, in Inches
Precipitation, in Inches

FIG. 3.

Inasmuch as maximum vapor pressure is a function of the temperature—doubling for every increase of approximately 18° Fahr.—the actual pressure of vapor present in the atmosphere also becomes a function of the temperature when the relative humidity remains constant. In other words, the rate of evaporation is approximately doubled for every 18° rise in temperature for constant humidity and wind velocity. Within the range of annual variation in temperature prevailing throughout the Northwest, the rate of evaporation will vary about 700 to 1 200%, due to temperature changes alone.

Figs. 4 and 5 show graphically the monthly mean relative humidity at a number of widely distributed points throughout the United States, outside of the arid region of the West. It will be noted that the monthly values in any State vary by only about 15 to 20% during the open season. Other conditions remaining constant, evaporation would vary only an equal amount.

The effect of wind on evaporation has been given various weights by different writers on the subject. Wielenmann, Stelling, and Tate, hold that evaporation varies approximately directly as the wind velocity. DeHeen, Shierbeck, and Svenson, hold that it varies as the square root of the wind velocity. Russell found a wind factor which, for wind velocities up to 15 or 20 miles per hour, could be represented approximately by the equation $1 + \frac{w}{4}$. FitzGerald found a wind factor represented by the equation $1 + \frac{w}{2}$. Bigelow, in his first evaporation formula, published in 1908, used a wind factor of about $1 + \frac{w}{35}$. Three years later he changed this to about $1 + \frac{w}{11}$. The writer believes that these formulas, giving the effect of wind on evaporation, refer to wind velocity at the elevation of the water surface in the basin from which evaporation was measured. The wind velocity, as observed by the Weather Bureau, is, in general, about three times the wind velocity at the surface of the earth.

In view of the discordant conclusions reached by the above mentioned experimenters with respect to the effect of wind on evaporation, the writer believes that no particular refinement is justified in selecting a wind factor. After checking observed and computed evaporation at a number of stations, using various wind factors, the

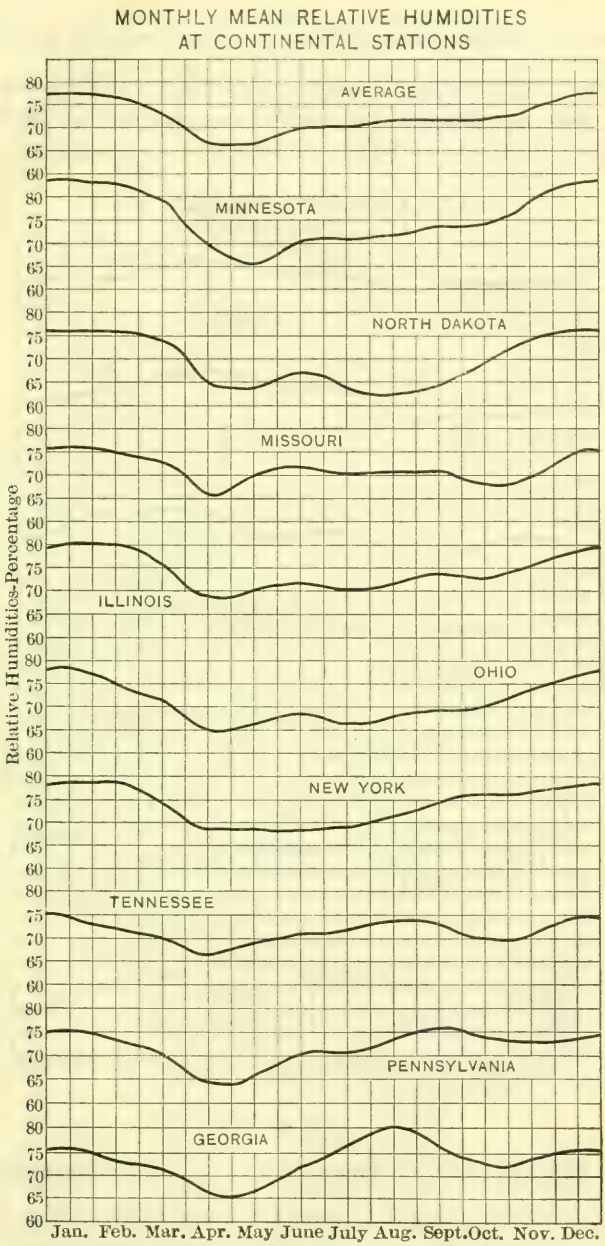

MONTHLY MEAN RELATIVE HUMIDITIES
AT CONTINENTAL STATIONS
AVERAGE
MINNESOTA
NORTH DAKOTA
MISSOURI
ILLINOIS
OHIO
NEW YORK
TENNESSEE
PENNSYLVANIA
GEORGIA
Relative Humidities-Percentage
80
75
70
65
60
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 4.

writer has tentatively adopted $1 + \frac{w}{10}$ as representing the effect of wind on the evaporation of water from surfaces at the level of the ground, where w represents wind velocity as observed by the Weather Bureau about 30 ft. above the general level of the surrounding country.

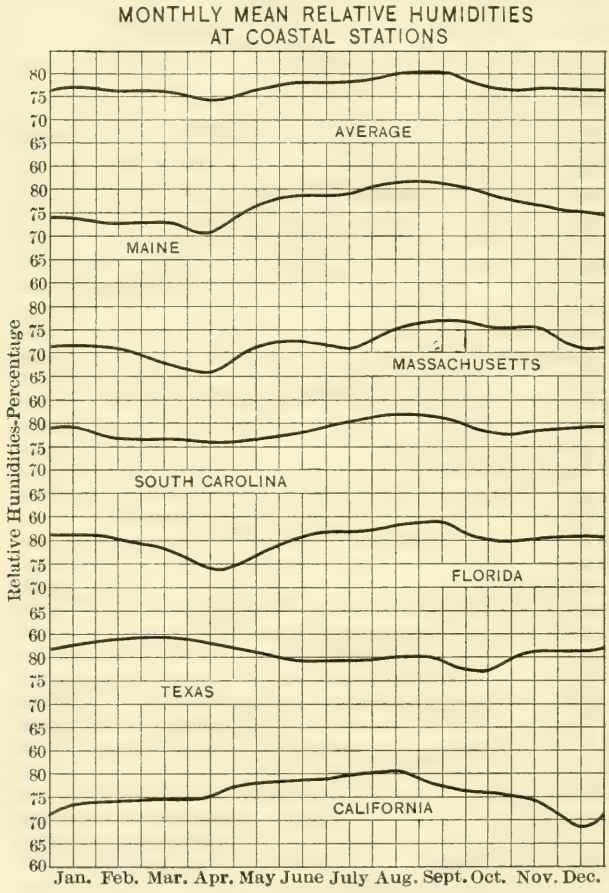


FIG. 5.

Using the writer's wind factor of $1 + \frac{w}{10}$, the annual variation in evaporation due to changes in monthly mean wind velocity amounts to only about 20 to 30 per cent. The monthly mean wind velocity

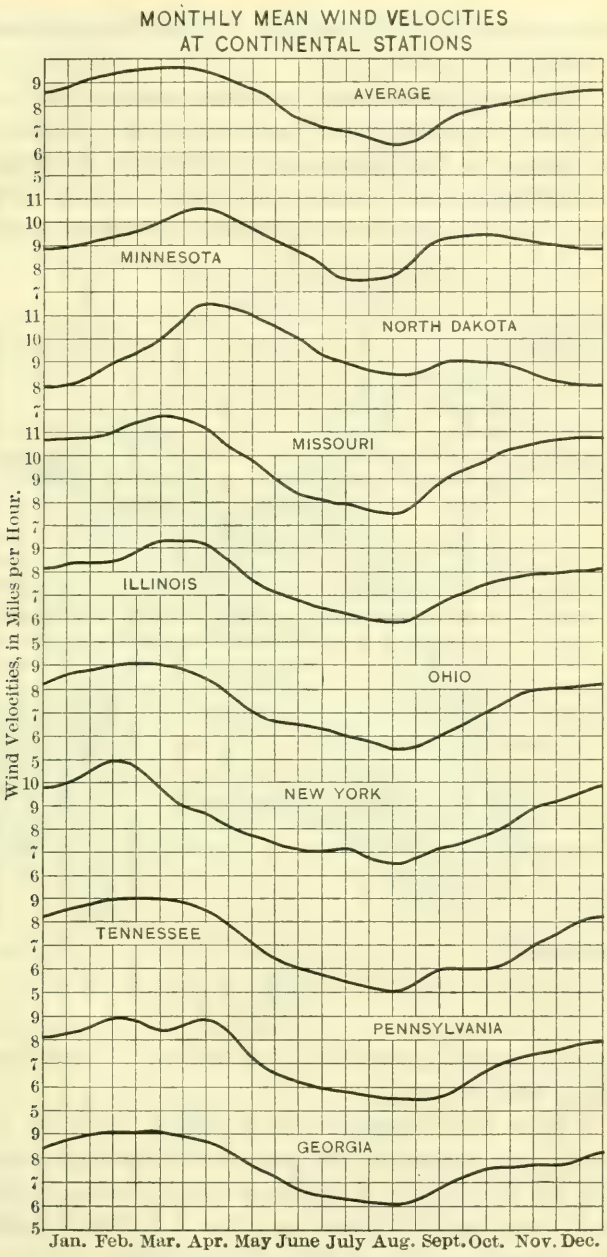

MONTHLY MEAN WIND VELOCITIES
AT CONTINENTAL STATIONS
AVERAGE
MINNESOTA
NORTH DAKOTA
MISSOURI
ILLINOIS
OHIO
NEW YORK
TENNESSEE
PENNSYLVANIA
GEORGIA
Wind Velocities, in Miles per Hour.
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 6.

at a number of stations in the eastern two-thirds of the United States is shown graphically in Fig. 6.

Fig. 7 shows the monthly mean relative humidity, wind velocity, air temperature, maximum vapor pressure, and the actual pressure of vapor present in the atmosphere at St. Paul, Minn. Neither relative humidity nor wind velocity, it will be observed, varies between wide limits. The vapor pressures and their differences, that is, the

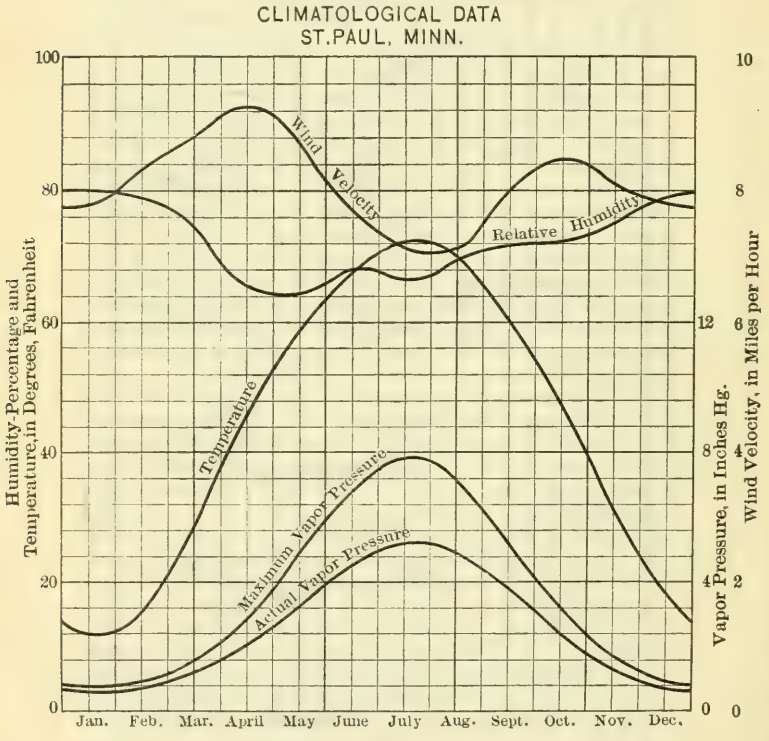


FIG. 7.

factor which virtually determines evaporation, all show variations of several hundred per cent.

Fig. 8 shows the evaporation from shallow water for different temperatures and seasons of the year, under conditions prevailing throughout the Northwest. A fundamental error has sometimes been made in plotting evaporation records, owing to failure to differentiate between the observed values at the different seasons of the year, all

months being thrown together indiscriminately and evaporation plotted against temperature.

By plotting separate values of evaporation for equal spring and fall temperatures, humidity and wind velocity for the given locality are properly taken into account.

Both the Grand River Lock and the University, North Dakota, evaporation stations are on small shallow bodies of water. The mean relative humidity at the North Dakota station is slightly less, and

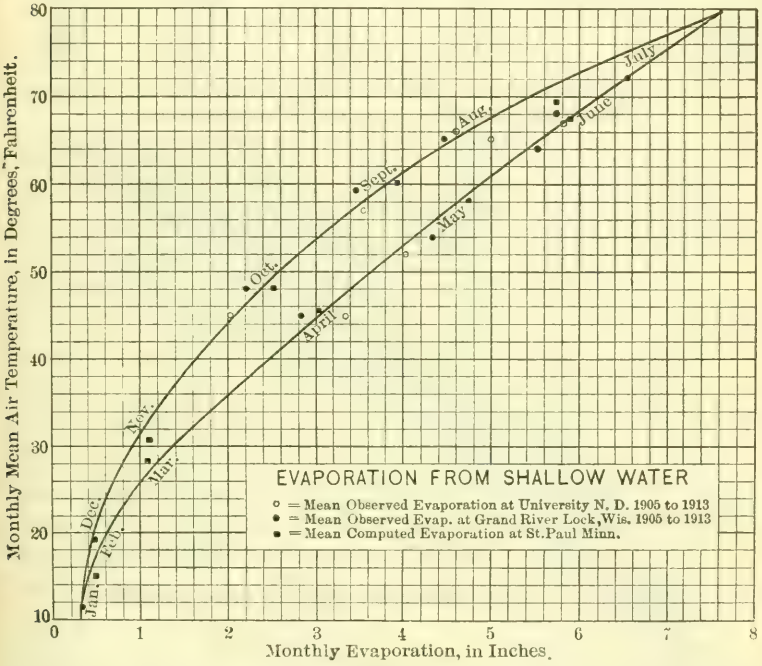


FIG. 8.

the wind velocity slightly greater than at the Wisconsin station. Monthly observations have been grouped together and the average taken. Observations of temperature and evaporation extend over the years, 1905 to 1913, inclusive. These values have been averaged by months, and the results are shown in Table 1.

During several successive months of either high or low precipitation a marked change in relative humidity is noticeable. Fig. 9, showing the effect of high and low rates of monthly precipitation on evaporation, is computed from meteorological data for St. Paul, Minn. The

TABLE 1.—OBSERVATIONS OF TEMPERATURE AND EVAPORATION.

Month.	At University, North Dakota.		At Grand River Lock, Wisconsin.	
	Air temperature, in degrees Fahrenheit.	Evaporation, in inches per month.	Air temperature, in degrees Fahrenheit.	Evaporation, in inches per month.
April	45	3.37	45	2.83
May	52	4.03	54	4.35
June	65	5.00	64	5.52
July	67	5.81	68	5.74
August	66	4.60	65	4.46
September	57	3.57	59	3.45
October	45	2.02	48	2.22
November	*34	*0.52	*41	*1.09

*Observations extended over portions of the month only.

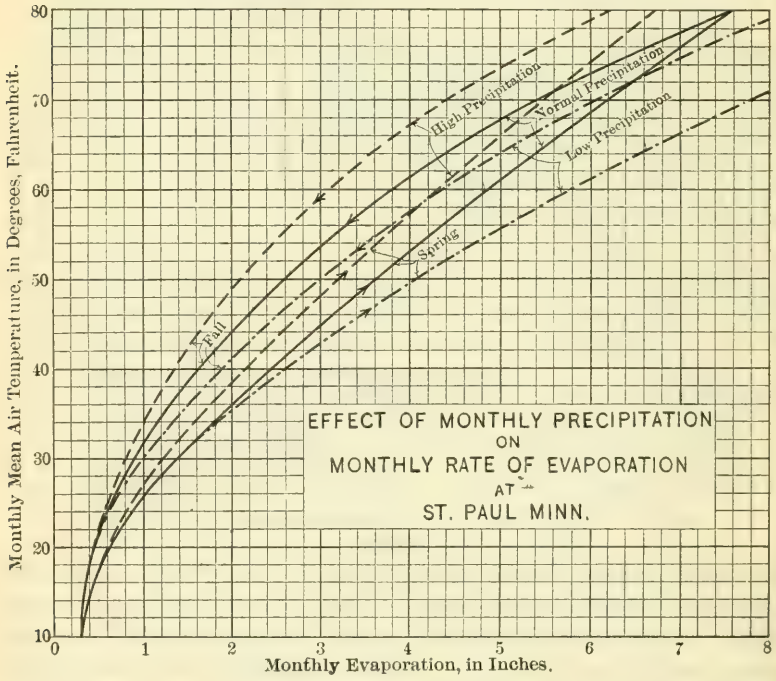


FIG. 9.

effect shown must be considered more or less approximate, because based on relatively few data, yet indicative of the variation in monthly evaporation resulting from large variations in monthly precipitation, and hence basic to the later discussion pertaining to the proportion

of precipitation lost by evaporation from land areas, at any given temperature, for various rates of rainfall. Both higher and lower than normal rates of precipitation, in inches per month, seem to affect the rate of evaporation less in early spring than in fall. This might be expected because the moisture available for evaporation in the spring is less dependent on rainfall than it is in the fall, hence the relative humidity would not be affected by changes in precipitation as readily in spring as in fall.

The evaporation at St. Paul, Minn., was computed by the following formula:

$$E = 15\ (V - v)\left(1 - \frac{w}{10}\right)$$

in which E = Evaporation, in inches per month;

V = Maximum vapor pressure, in inches of mercury, at monthly mean air temperature;

v = Actual vapor pressure, at monthly mean air temperature and relative humidity;

w = Wind velocity, in miles per hour, as measured by the Weather Bureau, approximately 30 ft. above the general level of the surrounding country.

The data and computations are given in Table 2.

In order to show a further comparison between actual and computed evaporation, the records for Mount Hope, N. Y., have been used. The observed data and the computations for this station are given in Table 3.

The data cover a varying period beginning between 1891 and 1896, and ending in 1912.

For January, February, March, October, November, and December, the evaporation was computed by taking the difference between the maximum vapor pressure, corresponding to water temperature, and subtracting an assumed vapor pressure in the air above the water, based on the observed relative humidity and water temperature.

From April to September, inclusive, the evaporation was computed by taking the difference between the maximum vapor pressure, corresponding to water temperature, and the actual vapor pressure in the air as determined from the air temperature and the observed relative humidity.

TABLE 2.—COMPUTED EVAPORATION AT ST. PAUL, MINN.

Month.	Temperature.*	Maximum vapor pressure in Hg.	Relative Humidity.		Difference in vapor pressure.	Wind velocity, in miles per hour.§	Precipitation.‖	Computed evaporation, in inches.
			Mean of 8 A. M. and 8 P. M.†	Mean.‡				
January...	11.6	0.069	80%	81	0.013	7.8	0.90	0.35
February..	15.0	0.081	79	79	0.017	8.3	0.84	0.47
March.....	28.2	0.151	74	75	0.038	8.8	1.60	1.07
April.....	45.7	0.306	65	66	0.104	9.3	2.33	3.02
May.....	58.2	0.485	64	65	0.170	8.7	3.12	4.76
June.....	67.4	0.670	68	67	0.221	7.7	4.41	5.88
July.....	72.1	0.786	66	¶67.5	0.255	7.1	3.40	6.55
August....	69.5	0.720	69	69	0.223	7.1	3.46	5.72
September	60.3	0.523	72	72	0.146	8.0	3.42	3.94
October....	48.1	0.335	72	73	0.091	8.5	2.34	2.53
November.	30.9	0.171	75	76	0.041	8.1	1.30	1.11
December.	19.3	0.099	79	81	0.019	7.8	1.06	0.51
Annual....	43.9	.....	72	.....	.....	.....	28.68	.....

* Mean of 43 years, 1871 to 1913, inclusive.
† Mean of 24 years, 1888 to 1911, inclusive.
‡ As modified by Minneapolis records.
§ Mean of 36 years, 1873 to 1908, inclusive.
‖ Mean of 43 years, 1871 to 1913, inclusive.
¶ As modified by Minneapolis, Moorhead, and LaCrosse records.

TABLE 3.—COMPUTED EVAPORATION, MOUNT HOPE, N. Y.

Month.	Temperature.		Relative humidity.	Maximum Vapor Pressure.		Observed vapor pressure in air.	Difference in Vapor Pressure.		Wind velocity.*	Precipitation.	Computed evaporation.	Actual evaporation.
	Air, in shade.	Water, in tub.		Water temperature.	Air temperature.		Air temperature.	Water temperature.				
Jan.....	24.8	32.5	78.0	0.183	0.129	0.101	0.028	0.082	10.0	1.83	1.21	1.27
Feb.....	23.3	32.3	77.9	0.182	0.120	0.092	0.028	0.090	10.9	1.49	1.26	1.26
Mar....	34.1	35.8	73.9	0.209	0.196	0.145	0.051	0.064	9.8	1.96	1.63	2.35
Apr....	46.7	46.6	68.3	0.317	0.318	0.217	0.101	0.100	8.8	2.21	2.82	2.97
May....	60.1	59.0	68.1	0.499	0.519	0.354	0.165	0.145	7.8	2.69	3.88	3.64
June....	70.5	68.2	68.1	0.689	0.744	0.507	0.237	0.182	7.2	2.47	4.70	4.40
July....	74.7	72.6	68.8	0.799	0.858	0.590	0.268	0.209	7.2	3.70	5.40	5.11
Aug....	72.1	71.3	71.3	0.765	0.786	0.560	0.226	0.205	6.5	2.91	5.08	4.73
Sept....	65.7	65.6	74.6	0.629	0.631	0.471	0.160	0.158	7.2	2.44	4.08	3.63
Oct.....	53.1	53.8	75.2	0.414	0.404	0.304	0.100	0.110	7.7	2.37	2.73	2.65
Nov	39.9	42.5	76.6	0.271	0.246	0.188	0.058	0.083	8.9	1.97	1.80	1.70
Dec.....	28.9	34.3	77.9	0.197	0.156	0.122	0.034	0.075	9.5	1.74	1.28	1.56
Annual.	49.5	51.2	73.2	.....	.....	.....	.....	.....	.....	27.77	.....	.....

* Wind velocity according to U. S. Weather Bureau at Rochester, N. Y., 1895 to 1906.

During the winter months the water in the floating tub was at a surprisingly high temperature, presumably due to the pumping of water into the reservoir in which the tub was floating. It is assumed, therefore, that the layer of air immediately above the water surface was heated to nearly water temperature, and that the most probable value for actual vapor pressure would be secured by applying the relative humidity to the maximum vapor pressure at the water tem-

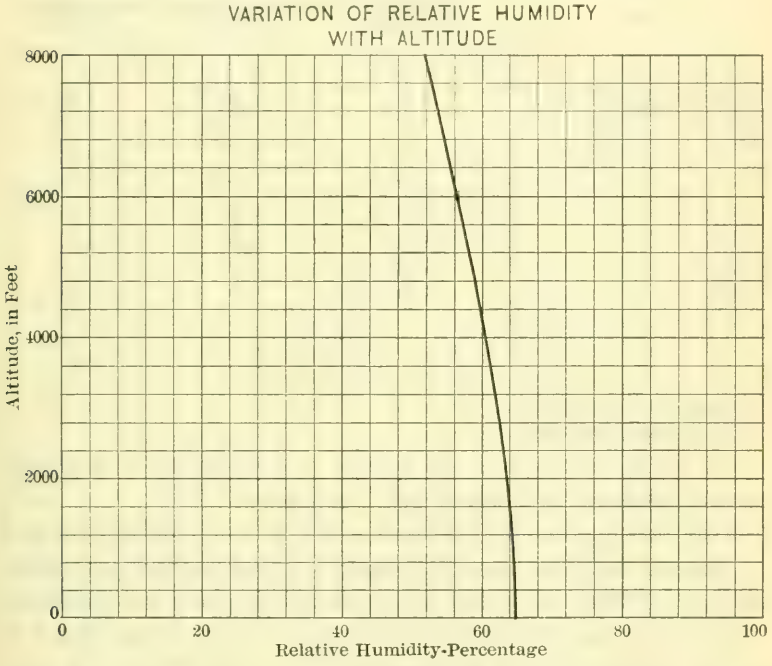


FIG. 10.

perature rather than at the air temperature observed some distance above the water surface. It is a fact, illustrated in *Fig. 10, that the average relative humidity varies only very slightly with changes of altitude of several thousand feet. This might be expected, because there is constant diffusion of vapor tending toward equal relative humidities. Tate says that vapor diffuses through the atmosphere much more rapidly than it is formed from the surface of the water, assuming no convection currents present. This rapid diffusion of

* Data from *Bulletin F*, U. S. Weather Bureau, 1898.

vapor, even in spite of air currents, must tend strongly toward the maintenance of equal relative humidities at all altitudes.

Table 4 gives the monthly evaporation* as observed and estimated by Desmond FitzGerald, Past-President, Am. Soc. C. E., at Boston, Mass.

TABLE 4.—Evaporation at Boston, Mass.

Month.	*Temperature, in degrees, Fahrenheit.	Observed values, 1875-1885.	Observed values, 1875-1890.
January	27	0.90†	0.96†
February	28	1.20†	1.05†
March	34	1.80†	1.70†
April	44.5	3.10†	2.97†
May	58	4.61	4.46
June	67	5.86	5.54
July	71	6.28	5.98
August	69	5.49	5.50
September	62	4.09	4.12
October	52	2.95	3.16
November	40	1.63	2.25
December	30	1.20†	1.51
		39.11	39.20

* Values taken from curve.

† Values largely estimated.

The values in Table 4 are platted in Fig. 11. The similarity between this curve and that of Fig. 8 is apparent.

Many other records of evaporation are available, but are omitted here because they do not give the evaporation from shallow water under conditions of wind and humidity prevailing throughout the Northwest.

The curves in Fig. 12 give the evaporation from open water in lakes of medium size and depth, and also the evaporation from snow and ice surfaces. The high rate of evaporation from snow under certain exceptional conditions has frequently been commented on. The values given are believed to be approximately right for ordinary conditions. The break in the up-going curve results from the fact that relatively large bodies of water remain frozen in spring until the monthly mean temperature has reached about 38 degrees. Until the break-up the evaporation is from snow and ice surfaces, and consequently larger, at the same temperature, than that from the relatively cold water immediately after the break-up. Most bodies

* *Transactions*, Am. Soc. C. E., Vols. XV and XXVII.

of water of moderate size freeze up in the fall, when the monthly mean temperature reaches about 20 degrees.

Horton gives some observations on evaporation from snow* which check the values indicated by the curves of Fig. 12 very well. A loss of 0.25 in. was recorded during the 9 days from December 26th, 1913, to January 4th, 1914. The mean maximum temperature during this

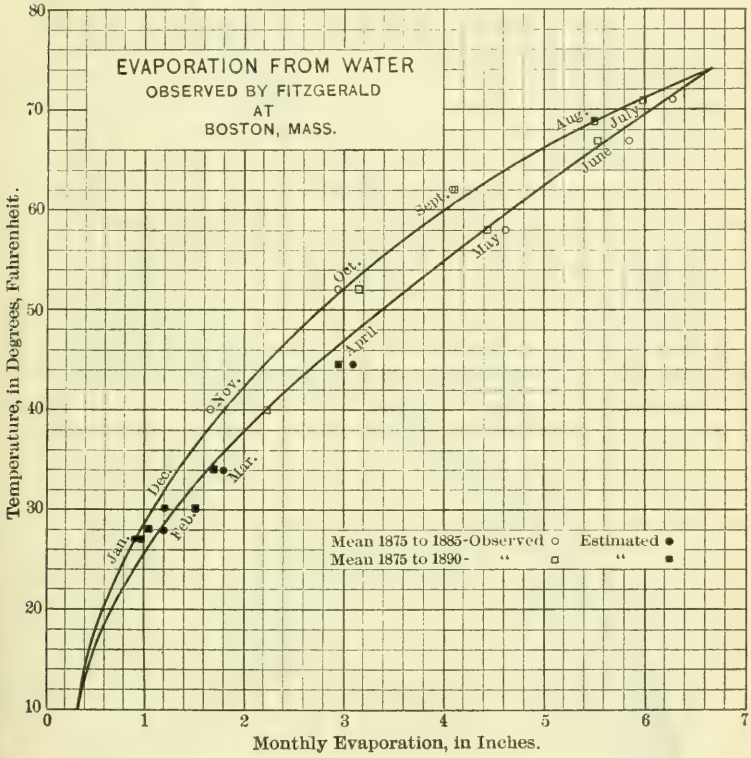


FIG. 11.

period was 29.5°, corresponding to a monthly mean temperature of about 24 degrees. The recorded evaporation for 9 days corresponds to a monthly evaporation of 0.83 in. The curve of Fig. 12 gives 0.9 in. of evaporation per month at a temperature of 24° under base conditions of wind and humidity. The mean wind velocity during the period of observation was 7.3 miles per hour. The relative humidity

* *Monthly Weather Review*, February, 1914.

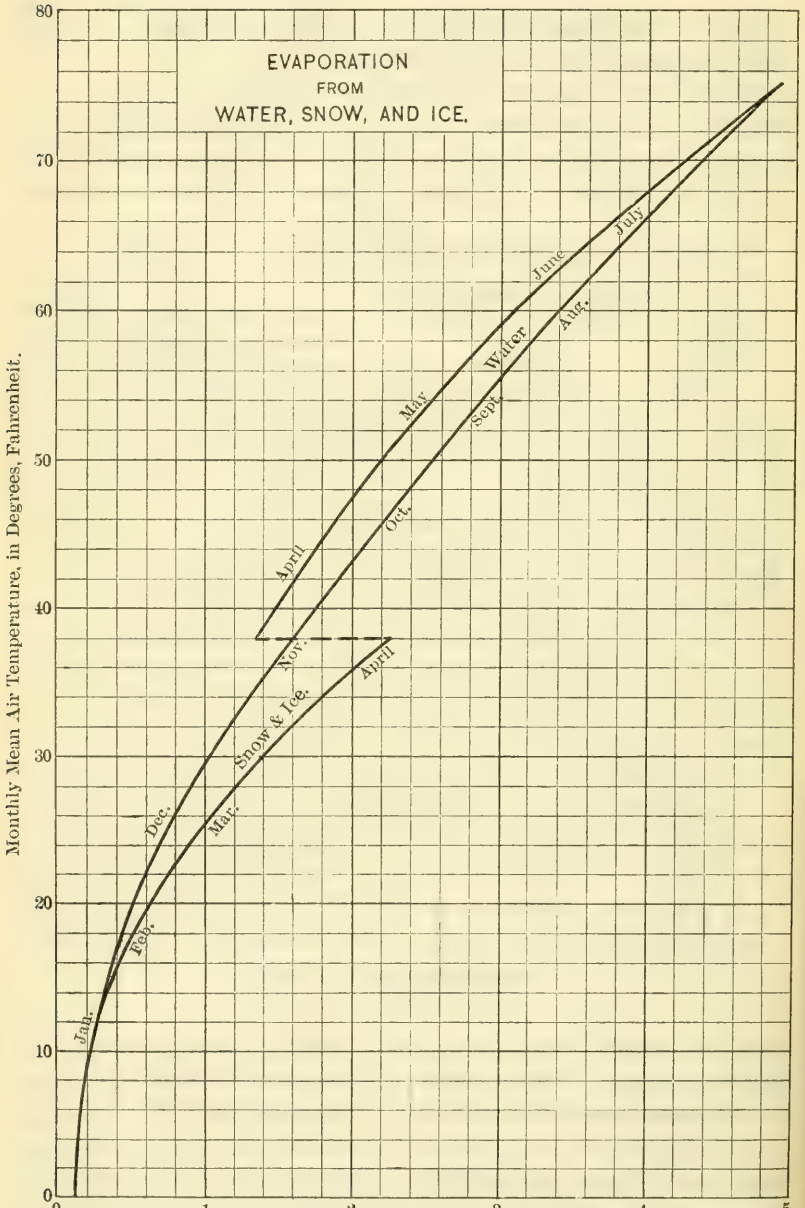

EVAPORATION
FROM
WATER, SNOW, AND ICE.
Monthly Mean Air Temperature, in Degrees, Fahrenheit.
Monthly Evaporation, in Inches.
0
10
20
30
40
50
60
70
80
1
2
3
4
5
Jan.
Feb.
Mar.
April
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Snow & Ice.
Water

FIG. 12.

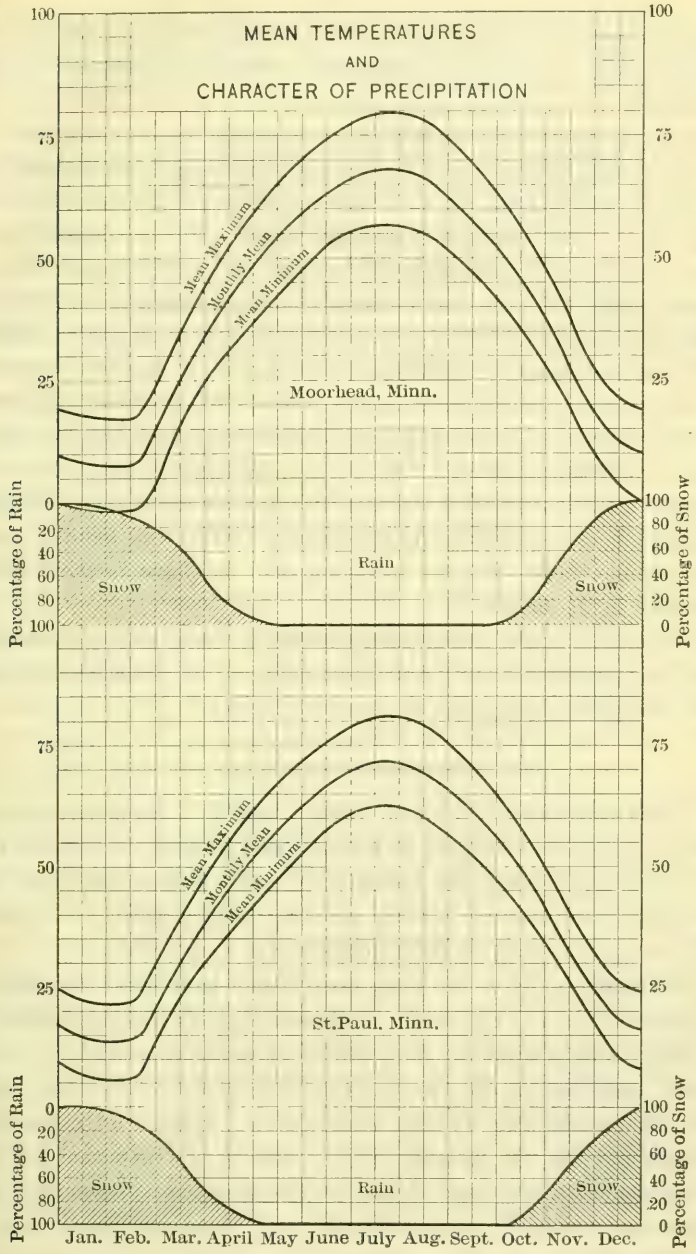

MEAN TEMPERATURES
AND
CHARACTER OF PRECIPITATION
Mean Maximum
Monthly Mean
Mean Minimum
Moorhead, Minn.
Rain
Snow
Snow
Percentage of Rain
Percentage of Snow
St.Paul. Minn.
Jan. Feb. Mar. April May June July Aug. Sept. Oct. Nov. Dec.

FIG. 13.

recorded by the Weather Bureau for January, 1914, was 78 per cent. These values represent approximately normal conditions for the Northwest.

Fig. 13 shows the monthly mean, and the mean maximum and minimum temperatures, and the approximate percentage of total precipitation which falls as snow during the various months of the year, at St. Paul and at Moorhead, Minn. For a monthly mean temperature of 23°, the mean of the maximum daily temperatures is approximately 32°, and for a monthly mean temperature of 41°, the mean of the minimum daily temperatures is approximately 32 degrees.

A certain quantity of thawing will occur at midday, when the monthly mean temperature reaches 18 to 20°, consequently, small patches of ground in the open fields will become bare and the evaporation from the water-shed will be reduced accordingly. When the monthly mean temperature reaches 32°, the maximum will be above freezing almost every day in the month. When the monthly mean temperature reaches 48°, the minimum will be above freezing almost every day in the month.

About 30% of the precipitation occurs as snow when the monthly mean temperature is 40°, and practically all of it occurs as snow when the mean temperature is below 20 degrees.

Evaporation from Land.

As the evaporation from land areas is usually a far more important factor in the determination of run-off from a given water-shed than that from water areas, the variation of such evaporation from land areas with temperature, season, rainfall, vegetal cover, topography, soil, and subsoil, must be determined, as far as possible.

The quantity of water evaporated from land areas depends on the rates of loss and the length of time during which evaporation can continue, as determined by the quantity of moisture available. Immediately following a rain storm, the rate of loss from land areas approximates the rate of loss from shallow water. As evaporation proceeds and the free moisture on the surface of vegetation and bare earth disappears, the rate of loss gradually becomes lessened. This rate of reduction of evaporation is more rapid at higher than at lower temperatures, so that though at first, after a rain storm, the

rate of evaporation from land varies with temperature as the rate of evaporation from water, it does not follow that the quantity evaporated per month is directly proportional to such rate of evaporation.

Experiments made in the West by the U. S. Department of Agriculture, in connection with its irrigation investigations, indicate that the temperature of dry soil in the sun is higher and the temperature of moist soil is lower, in summer, than that of the overlying air. The assumption used herein, that the temperature of the moisture on the surface of the ground, on the surface of vegetation, and in the upper layers of the soil, is on an average, approximately equal to that of the air, is believed to be sufficiently close to the truth for present purposes. The monthly mean air temperature, then, becomes an index to the monthly mean rate of evaporation from land areas. It also virtually determines the quantity evaporated per month whenever there is sufficient precipitation to keep the ground saturated, or covered with snow.

Fig. 14 shows the relation between the moisture available in the soil and the quantity evaporated.* The soil experimented with was a well pulverized, sandy loam. The rapid decrease in the rate of evaporation after the percentage of moisture dropped below 10 is significant. When the moisture content of this particular soil dropped below 3.5% of its dry weight, evaporation practically ceased.

Fig. 15† shows the relation between "irrigation" and "evaporation". The irrigation water was applied twice a month, thus roughly approximating rainfall conditions. The constant of evaporation in the given case (the moisture content at the beginning of the experiment was 7.73%) was 0.15 in. For an irrigation of 1 in. per month, that is, for an approximate equivalent of 1 in. of rainfall per month, the evaporation was 0.15 in. plus 95% of the "rainfall". For an approximate equivalent of 2 in. of rainfall the evaporation was 0.15 in. plus 80% of the "rainfall". For 3 in. it was 0.15 in. plus 74% of the "rainfall". For 4 in. it was 0.15 in. plus 71% of the "rainfall". These facts again indicate that the evaporation loss varies much more rapidly at low

* The data used in constructing this curve were taken from *Bulletin No. 177*, Office of Experiment Stations, U. S. Department of Agriculture.

† Also based on data secured from *Bulletin No. 177*, Office of Experiment Stations, U. S. Department of Agriculture.

than at the higher rates of rainfall. The recorded evaporation from a water surface averaged 10.9 in. during the period of the experiment from which the foregoing data were obtained.

It may here be remarked that, for an air temperature of 85°, corresponding to the air temperature for the experiment just quoted, the writer's curves of "Evaporation from Land Areas", shown later in Fig. 19, indicate a maximum evaporation of 7.3 in. from saturated soil under base conditions. Under Tulare, California, conditions, agreeing with those of the foregoing experiment with respect to humidity, bare soil, and flat slope, a constant of 1.4 to 1.5 would be applied to the base values, giving from 10 to 11 in. as the maximum evaporation from a saturated soil surface under the given conditions. These base curves of evaporation, Fig. 19, indicate 15 in. of rainfall per month as permitting of maximum evaporation. The equation for the curve of evaporation, shown in Fig. 15, is 0.5 in. plus 62% of the "irrigation" (approximately "rainfall"). Applying this formula to 15 in. of rainfall gives 9.8 in. as the monthly evaporation.

Inasmuch as the application of irrigation water every 2 weeks only roughly approximates the condition of natural precipitation, the curve of Fig. 15 must be considered as merely indicative of how evaporation varies with rainfall.

The quantity of moisture available for evaporation, after a given fall of rain on any particular water-shed, depends mainly on the character and rate of precipitation, the quantity and kind of vegetal cover, the character of the soil, the slope of the ground, the temperature, and the season of the year. Torrential rains permit less evaporation than an equal quantity of precipitation in scattered showers. Snowfall permits relatively greater evaporation than rainfall.

The character of precipitation is well indicated by the temperature, and the hourly and daily rate of precipitation in any one season averages approximately the same from year to year in any locality, when a few exceptional storms are eliminated. It is realized, of course, that these exceptional storms are usually the ones which determine flood flow, particularly on small streams, even though they may not have material bearing on the annual run-off. Where the hourly and daily rates of rainfall are known to be radically different for two water-sheds, this factor can be taken into account in computing run-off, as will be explained later.

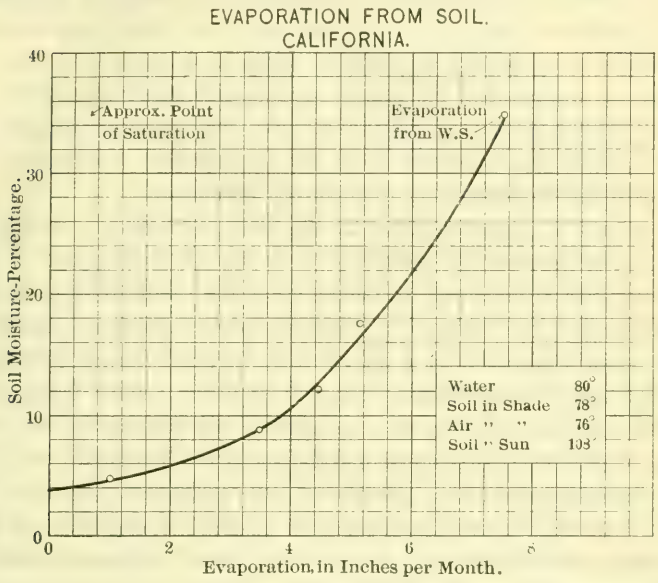

EVAPORATION FROM SOIL.
CALIFORNIA.
Approx. Point of Saturation
Evaporation from W.S.
Water 80°
Soil in Shade 78°
Air " " 76°
Soil " Sun 103°
Soil Moisture-Percentage.
Evaporation, in Inches per Month.
0
10
20
30
40
2
4
6
8

FIG. 14.

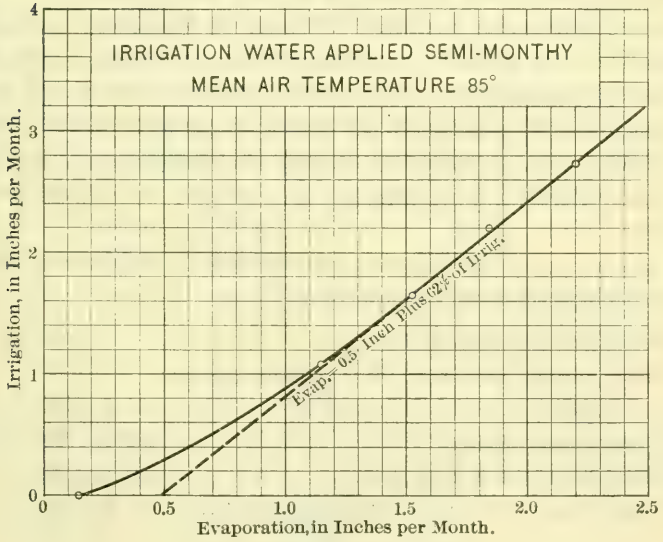

IRRIGATION WATER APPLIED SEMI-MONTHY
MEAN AIR TEMPERATURE 85°
Evap. + 0.5 Inch Plus 62% of Irrig.
Irrigation, in Inches per Month.
Evaporation, in Inches per Month.
0
1
2
3
4
0.5
1.0
1.5
2.0
2.5

FIG. 15.

The character and quantity of vegetation on a given water-shed have a bearing on the loss out of rainfall from that water-shed, in that vegetal cover affects evaporation from land areas, and that the quantity of water used by plants varies with their character and growth. The quantity of rain and snow which never reaches the ground but is re-evaporated from the surfaces of vegetation also varies with the character and quantity of vegetation.

Unfortunately, experimenters in this field have usually stated the quantities intercepted by vegetation in terms of "percentage of precipitation". It is apparent, of course, that the quantity of precipitation intercepted by leaves, etc., depends largely on the quantity of rain or snow that falls in a given time, a much larger percentage being lost out of small showers than out of heavy rains. When moist snow falls on conifers, very large quantities are intercepted and more rapidly evaporated than if they had fallen to the ground in the open. Where water-sheds with a growth of only deciduous vegetation are under consideration, the distribution of rainfall throughout the year, particularly with reference to the growing season (although these are often coincident), also affects the quantity intercepted by such vegetation. In a general way, the interception by different forms of vegetation has been placed at from 12 to 30% of the rainfall.

It is a well-known fact that all forms of vegetation, particularly forests, shade the ground to a certain extent, and consequently reduce the rate of evaporation of free moisture. Whether or not they reduce the total quantity evaporated per month or per year depends also on the relative rates at which the rainfall can percolate into the forest floor, the cultivated field, and the bare ground, or run off into the streams.

Transeau* gives the following relative rates of evaporation observed at Cold Spring Harbor, Long Island:

Bare sand and gravel slide	100 per cent.
Open garden plot with low herbaceous vegetation	80 to 100 " "
Upper beach areas	80 to 90 " "
Light forest on gravel soil	50 to 70 " "
Dense forest with abundant undergrowth	35 to 40 " "

* *Botanical Gazette*, April, 1908.

Dense ravine forest with abundant herbaceous vegetation	13 per cent.
Dense swamp forest with abundant undergrowth and water near surface	10 " "
Fresh-water marsh	45 " "

The evaporation was measured with porous-cup atmometers placed about 4 in. above the surface of the ground.

The loss of rainfall through evaporation (including reduced evaporation from the ground as counterbalanced by interception) from a dense deciduous forest has been assumed by the writer, for the present, as being about 15 or 20% less than the evaporation from an open grain field or grass land. Evaporation from a dense coniferous forest has been assumed as greater than that from an open field during the winter and less in summer. Evaporation from a brush-covered water-shed has been assumed as somewhat less than that from open fields.

The quantity of water available for evaporation from any given water-shed is also influenced by the rate at which the rainfall can percolate into the ground or run off into the streams. Water-sheds with irregular, rough surfaces, and steep slopes, permit rapid surface run-off. Flat, or gently sloping water-sheds, of course, permit less rapid run-off, and hence greater evaporation. Sandy soils permit high rates of percolation. Clayey soils retard percolation, and facilitate surface run-off when on steep slopes, increase evaporation when on flat slopes, have great moisture-holding capacity, and exert a large capillary lift in bringing moisture to the surface for evaporation.

King* found the following rates of percolation through columns of sand and soil having a cross-section of 0.1 sq. ft. and 14 in. long, when kept covered with 2 in. of water:

In No. 40† sand,	percolation at the rate of	301	in. per day.
In No. 100† "	" " " " "	39.7	" " "
In clay loam	" " " " "	1.6	" " "

Though it is very improbable that any but the most exceptional water-sheds of small area would have a surface covering of as coarse

* Nineteenth Annual Report, U. S. Geological Survey.

† No. 20 sand, effective diameter, 0.474 mm.
" 40 " " " 0.185 "
" 60 " " " 0.155 "
" 80 " " " 0.138 "
" 100 " " " 0.083 "

sand as No. 40, yet the differences between clayey and sandy soils, as to the facility with which rain water can percolate through them, is very marked.

The rate of percolation is also affected by the initial condition of the soil. When the upper layers become nearly air dry to any considerable depth, the pore space becomes so filled with air as to retard greatly the entrance of water. This is particularly true of the denser soils the individual pore spaces of which are relatively small, even though their moisture-holding capacity may far exceed that of the coarser sands. It has been found that $\frac{1}{2}$ in. of precipitation will moisten the upper layers of soil sufficiently to establish contact with the lower layers containing more moisture, and thus increase, through capillarity, the moisture content of the upper layers of soil by a far greater quantity than the precipitation received. In this way ground-water may be brought to the surface and evaporation losses increased.

Another factor which affects the total quantity of evaporation from a given water-shed is the depth to which the percolating waters pass, and the ability of each particular soil to raise water to the surface again by capillary action. Experimenting on a series of cylinders, each having an area of cross-section of 0.1 sq. ft., and filled with a mixture of sand in approximately natural proportions, grains varying in size from No. 100 to No. 20, King found the following rates of evaporation for capillary lifts varying from 6 to 30 in. The temperature of the air in the laboratory where the experiment was conducted was about 70° Fahr., and the relative humidity is reported to have been very low.

Capillary lift, in inches above ground-water table.	Evaporation, in inches per month.
6	3.42
12	3.34
18	2.39
24	1.04
30	0.58

It is worthy of note that the maximum evaporation given in the above tabulation is only about one-half of what might be expected from a free water surface under the conditions of temperature and humidity stated. It is probable that if evaporation had been accelerated, capillarity would have shown itself equal to raising the mois-

ture to the surface at a more rapid rate than that found by the experiment for capillary lifts of only a few inches.

The rates of evaporation given for a capillary lift of 30 in. indicate that when the water-table for the particular soil used in King's experiments drops to more than 4 or 5 ft. below the surface of the ground, evaporation is reduced to a very small quantity. This conclusion agrees with the statement of Charles H. Lee,* Assoc. M. Am. Soc. C. E., to the effect that capillary lift is practically limited to 4 ft. in coarse sandy soil, and to 8 ft. in fine sandy or clayey soil. It also agrees reasonably well with the conclusions of McGee,† stated in the following terms:

"While the effectiveness of capillary movement varies with the texture and structure of soil and subsoil and underlying rock, it may be said broadly that under average conditions capillarity acts freely to 4 or 5 feet in depth, fairly to 10 feet, and slowly to 30 or more feet."

Capillary action is facilitated by the rotted fibers of dead roots, which in some forms of vegetation penetrate to considerable depth.

In relation to ground-water, it is interesting to note that the "ground-water table" is not exactly what the word "table" might imply. On the basis of a detailed study of an area of about 1 200 by 1 800 ft., on the shore of Lake Mendota, Wisconsin, King found that the ground-water "table" presented undulations roughly conformable with the relief of the ground surface.

As the slope of the ground-water surface toward the stream affects the rate at which the ground-waters will be discharged into the stream, the rise in ground-water level after a given quantity of water has been allowed to percolate through soils of various moisture-holding capacities, has considerable bearing on the resulting increase in stream flow for any given quantity of percolation.

Fig. 16, based on King's experiments,‡ shows the quantity of water held by capillarity in coarse and fine sands at various heights above the water-table, after thorough drainage had been permitted for 2½ years. Evaporation from the surface was prevented. The greater moisture-holding capacity of fine sand is well illustrated by these curves.

In discussing the rise of ground-water level as the result of pre-

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 148.

† 1911 Year Book of the Department of Agriculture, p. 482.

‡ Nineteenth Annual Report, U. S. Geological Survey, pp. 67 to 294.

cipitation and percolation, King states that only about $\frac{1}{4}$ in. of water would be required to raise the ground-water level in the fine sand 1 ft., and that about $\frac{3}{4}$ in. would be required to raise that an equal amount in the coarse sand. This deduction relative to the great rise of ground-water level following a given percolation, and the difference in the rise in ground-water level in coarse and fine sands due to the same quantity of percolation, does not seem warranted. King appears to have reached his conclusion by computing the additional quantity of water required to produce saturation in the given sand up to a

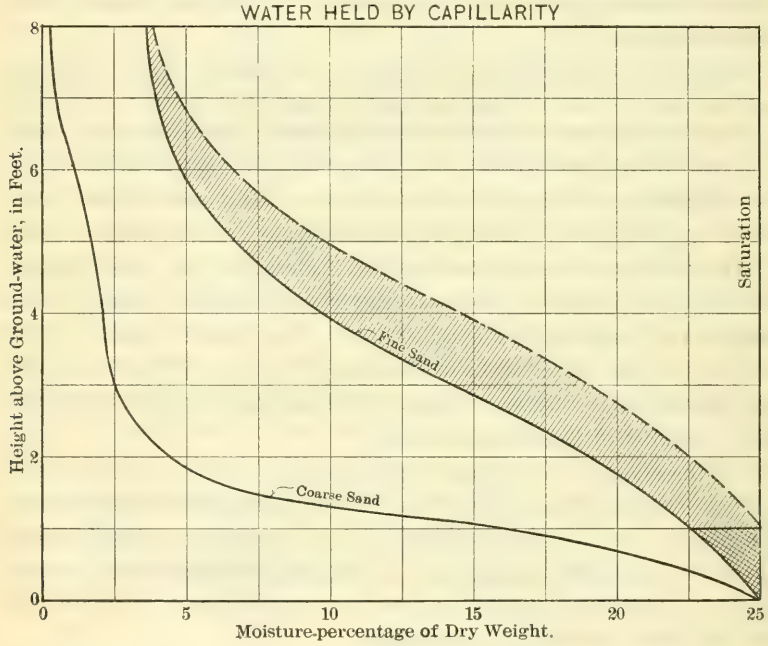


FIG. 16.

point 1 ft. above the water-table, assuming that no additional water would be lifted by capillarity above the original ground-water level. This reasoning is illustrated by the small triangular shaded area in Fig. 16. It appears to the writer, however, that the quantity of moisture required to raise the ground-water level 1 ft. would be indicated approximately by the lightly-shaded area between the broken and solid-line curves in Fig. 16, based on the assumption that the moisture held by capillarity is determined by the height above the ground-water table.

Experiments made recently at the University of Minnesota, by the writer, using fine sand, indicate that the foregoing conclusion is correct. Equal additions of water raised the point of saturation an increasing, instead of a decreasing, amount, as stated by King.

The shape of the curves shown holds under field conditions only when the point 8 ft. above the ground-water level is sufficiently far below the surface of the ground to be unaffected by surface evaporation. Under all other conditions the curves would be flattened.

It is probable that, under ordinary field conditions, 1 in. of rain soaking deeply into the ground will raise the ground-water level from 4 to 6 in., depending on the character and moisture content of the soil. A given addition to the ground-water will result in a greater increase in seepage flow to streams lying in clayey soils than in sandy soils, but the total ground-water supply to streams in sandy soils will be the greater because a larger percentage of rainfall is absorbed by such soils.

Transpiration.

Perhaps no more uncertain factor enters into the computation of run-off than the water requirements of growing plants. Most of the available data, relative to water consumption of plants, are inapplicable and so widely divergent as to be of little value. Laboratory results are usually based on experiments with single plants, with no indication of the ground space covered by such plants. Frequently, transpiration is compared with other phenomena, such as evaporation from water or soil surfaces or precipitation. The divergence of opinion with respect to transpiration is well indicated by the fact that Schleiden thought the transpiration from a forest was three times the evaporation from a water surface of equal area, whereas Schübler thought it only one-quarter, and Pfaff thought it to be 0.87 to 1.58. Extreme values given for annual transpiration vary from less than 1 in. to 16 ft. Most of the values given for forest trees and small grains, however, vary from 4 to 9 in. per year, with occasional values for oats and some grasses running up to 14 and 15 in. per year.

One experimenter found that shade reduced the transpiration of conifers but increased that of deciduous trees; others found that shade reduced transpiration of all plants experimented with by about one-half. In Cuba, Hasselbring found that the shade of ordinary cheesecloth reduced the transpiration of tobacco plants 30 per cent.

Transpiration, being the vaporization of water from the breathing pores or stomata of leaf and other vegetable surfaces, must necessarily depend largely on the same factors as evaporation. Most important among these is temperature. The law first stated by Van't Hoff and Arrhenius, that most chemical reactions and physiological processes double in activity for every rise in temperature of 10° cent., is quite firmly established. It has been found to be substantially correct for the rate of fixation of carbon dioxide by plants in sunlight; and, inasmuch as transpiration occurs during the process of carbon dioxide assimilation when the stomata open in the sunlight, it is reasonable to assume that the rate of transpiration, in so far as it is dependent on temperature, substantially follows Van't Hoff's law.

In applying this law, however, it is difficult to decide on a temperature at which plant activity begins. Köppen regards all monthly mean temperatures less than 48° as included in the period of rest of plants. Other scientists state that the protoplasmic contents of vegetable cells are inactive while the temperature is below 6° cent. (42.8° Fahr.). In temperate latitudes, when there is a lack of precipitation or irrigation, monthly mean temperatures of more than 72° constitute a period of summer rest for most plants. When sufficient moisture is present, they constitute a period of ripening for southern fruits, and, when there is an abundance of moisture, these high temperatures constitute the period of sub-tropical growth.

Botanists agree that every plant has its optimum temperature and moisture values, during which it makes its best development. When there is an excess of moisture, crop yields are determined largely by temperature. When rainfall is sufficient and the temperature is too low for best growth, sunshine becomes the most important factor. Heat cannot replace sunlight in the growth of vegetation, but sunlight can partly replace heat. When temperature and sunshine are sufficient, crop yields depend mostly on rainfall. Writing on the effect of water on the yield of corn, Smith says:*

"The grain plant obtains a great deal of its total weight from the soil during the early part of its growth, and a lack of moisture at this time will cause a short straw but not necessarily a small yield of grain. During the latter part of the growth the seed is being made chiefly from material stored in the stalk, and moisture must be present

* *Monthly Weather Review*, February, 1914.

to flush the material from the stalk into the head or the grain will be shrunken."

Fig. 17 shows the writer's "Base curve of Transpiration", founded largely on Van't Hoff's and Arrhenius' law. When the moisture supply is ample, mixed vegetation native to the Northwest will have a normal transpiration of about 10 in. per season. Taking off from the base curve, the transpiration, in inches per month, corresponding to the monthly temperatures prevailing in Central Minnesota, for example, and summing up these monthly values, gives a normal seasonal transpiration of practically 10 in. If, on the other hand, the monthly temperatures prevailing in Mississippi, for example, are used in determining monthly transpiration, the total seasonal transpiration—as determined by temperature alone, without consideration of the other factors which modify these values—in Mississippi would be about twice as much as in Minnesota, according to the base curve of Fig. 17. On some water-sheds where the moisture supply is always ample, monthly transpiration is dependent on temperature, sunshine, and the form of vegetation. On other water-sheds, such as those in Mississippi, the available supply of moisture is usually the governing factor, within proper temperature limits. The curve of Fig. 17, at best, can serve merely as one of the guides in arriving at the monthly distribution of a given quantity of seasonal transpiration. Unrestricted transpiration, within the limits of utilization of the given form of vegetation, occurs on few water-sheds even for all the months of the growing season in any year. A deficiency of rainfall in a given month, especially in spring, usually affects the transpiration for all the remaining months of that season, particularly in the case of trees and cultivated grains. Grasses respond more quickly to increased moisture supply. The monthly transpiration, then, will seldom be a constant for any water-shed from year to year, even for the same form of vegetation.*

* The application of Fig. 17 may be illustrated further, in this manner:

MISSISSIPPI RIVER WATER-SHED, 1901. TABLE 7.

Month.	Monthly Temperature.	Transpiration, off Curve.
April	41.7	0.2
May	57.1	1.5
June	62.1	1.9
July	69.9	2.4
August	65.2	1.9
September	55.1	1.0
October	46.7	0.3
Total		9.2

Similarly, the normal transpiration, as shown by the curve for the temperature prevailing on the same water-shed in 1907, gives 8.1 in. To indicate the great range

Most experimenters have found that the quantity of water transpired by plants varies approximately as the quantity of dry substance produced. Whether or not this relationship is purely accidental does not invalidate the fact. In the 1903 Year Book of the U. S. Department of Agriculture is given the average yield of corn for 15 years in the principal corn-growing States, together with the average precipitation over those States during June, July, and August. When platted, these data indicate an average yield of 5 + 2 bushels for every inch of rainfall during June, July, and August, between the limits of yields of 15 and 35 bushels per acre.

The data on water requirements of crops, recorded in *Bulletin 177,* Office of Experiment Stations, and *Bulletins 130, 188,* and *201,* Bureau of Plant Industry, U. S. Department of Agriculture, though not conclusive, indicate that the yield of grain is approximately proportional to the quantity of water consumed. The great difficulty encountered in the above mentioned experiments was that of differentiating between the evaporation from the soil and transpiration.

Livingston* gives considerable experimental data which show an almost direct relationship between transpiration and weight of vegetable substance produced.

In view of the substantially constant relationship for any given species of plant, which has been found by most experimenters between transpiration and vegetable substance produced, yields of hay, grain, etc., become a convenient index to the approximate relative quantities of transpiration to be expected on different water-sheds, and on the same water-shed in different years.

The ratio of water used to dry substance produced has been found to vary with individual plants and with the plant environment. Con-

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of seasonal transpiration due to temperature changes alone, similar values may be given for other water-sheds, as follows:

Name of Water-shed.	Year.	Total Seasonal Transpiration, off Curve.
Little Fork	1909	8.1
" "	1910	8.5
Minnesota	1909	10.3
"	1910	12.3
Ottertail	1908	9.8
"	1909	9.3
St. Croix	1907	7.5
" "	1912	9.8
Tombigbee	1906	20.1

It will be noted from these figures that, in a general way, using the mean monthly temperatures for water-sheds in Minnesota, the curve will give a normal seasonal transpiration of about 10 in. The curve, of course, takes into consideration only one factor, namely, temperature. Character and density of vegetation, hours of sunshine, available moisture, etc., all enter in determining transpiration.

* *Botanical Gazette,* Vol. 40, p. 31.

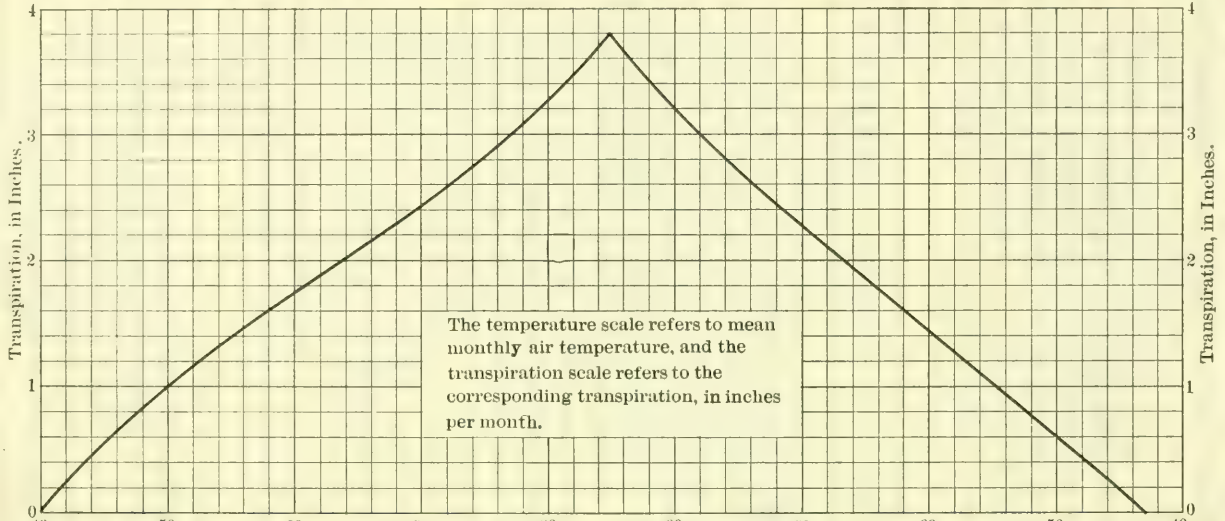

Transpiration, in Inches.
0
1
2
3
4
40
50
60
70
80
80
70
60
50
40
The temperature scale refers to mean monthly air temperature, and the transpiration scale refers to the corresponding transpiration, in inches per month.
Temperature, in Degrees, Fahrenheit.
March to July August to November
BASE CURVE OF TRANSPIRATION

FIG. 17.

ifers, in particular, have been found to use less than deciduous trees; in fact, they are said to use less than one-sixth as much. For grass and grain, the ratio of pounds of water used to pounds of dry substance produced seems to vary from about 300:1 to 600:1. Some writers say deciduous trees use more water than grasses and grains, and others claim just the opposite. Something seems to depend on each particular writer's view as to the effect of forests on climate, stream flow, and kindred subjects.

The total annual transpiration for mixed vegetation, as previously stated, seems to vary between 4 and 9 in.

Most students of the subject of transpiration seem to be agreed that the quantity of water used by plants during the growing season depends mainly on the quantity available within reach of the root system. If plants transpired a quantity of water equal to from one-half to two times the evaporation from an equivalent surface of water, as claimed by some experimenters, a great many streams in the United States that have a very appreciable sustained flow would become intermittent, because there would be no ground-water supply to feed them. Surface run-off alone would appear in these streams.

It has been found that in any given soil all forms of vegetation wilt when the moisture content is reduced to a certain percentage. This percentage, however—known as the "wilting coefficient"—varies greatly for different soils. Fig. 18 shows the wilting coefficient and the moisture-holding capacity for various soils, expressed in percentages of the dry weight of the soil. The area between the two curves represents graphically the maximum quantity of moisture, in percentages of dry weight, which different soils can hold available for plant growth. The curves are based on data secured from *Bulletin 230*, Bureau of Plant Industry, U. S. Department of Agriculture.

In view of the fact that the moisture-holding capacity was determined from short soil columns, 1 cm. in length, it does not represent the moisture-holding capacity of long columns of similar soils under field conditions of drainage, yet gives an indication of such relative capacities. It may be noted that the moisture-holding capacity of sand agrees with similar values taken from the curves of Fig. 16, at the point of saturation.

Wilting coefficients and moisture-holding capacities, when expressed in percentages of dry weight, as is usually done, give a

somewhat distorted impression as to the relative capacities of sand and clay, because of differences in the dry weight of these materials. The actual maximum quantity of water available for plant growth varies from about 3.75 in. per ft. of depth of sand to 6.5 in. per ft. of depth of heavy clay, or about one-half the difference shown between these materials when available water is expressed in percentages of dry weight, as is usually done.

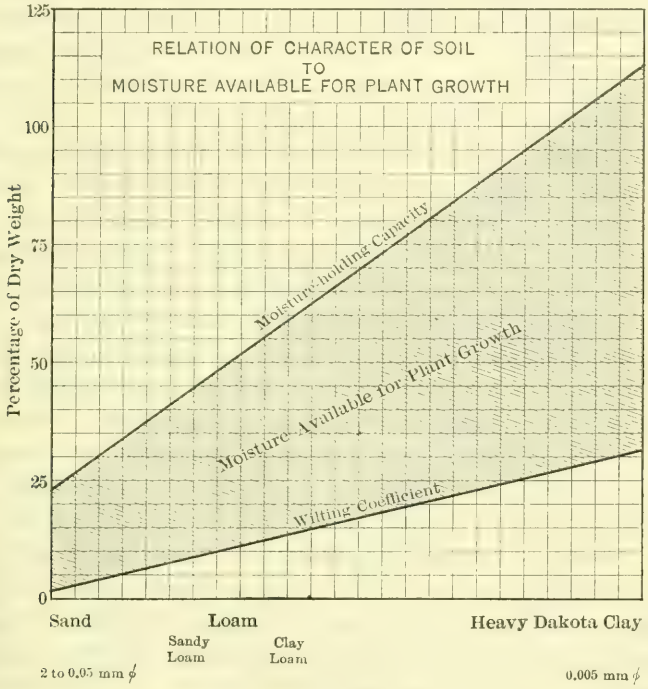


FIG. 18.

The true quantity of moisture available for plant growth under field conditions would seem to be given by the difference between the wilting coefficient and the moisture held in different soils by capillarity when free drainage is permitted, plus the quantity which can be absorbed by the plants during rains and before the excess moisture has had time to percolate into the subsoil out of reach of the root system. Expressed in inches of depth over the surface, the moisture available for plant growth after drainage under field con-

ditions, would appear to vary from about 1 to 2 in. per ft., depending on the character of the soil.

Even though all plants have been found to wilt when the moisture content for a given soil has been reduced to a certain percentage, the fact must not be lost sight of that in most fields the character of the soil varies greatly from foot to foot of depth, and that the roots of different forms of vegetation penetrate to widely varying depths, usually adapting themselves, in a measure, to the available moisture content of the various soil layers. Frequent light sprinkling of lawns is well known to make the grass non-drought-resistant, because it coaxes the root system to the surface where the greatest supply of moisture is temporarily found.

Transpiration of deep-rooted vegetation will be less fluctuating with varying monthly and annual rainfall than that of shallow-rooted vegetation. In dry seasons deep-rooted vegetation will draw more on ground storage. The transpiration of plants growing in sandy soils will vary more with rainfall than that of similar plants growing in clayey soils, other conditions remaining the same.

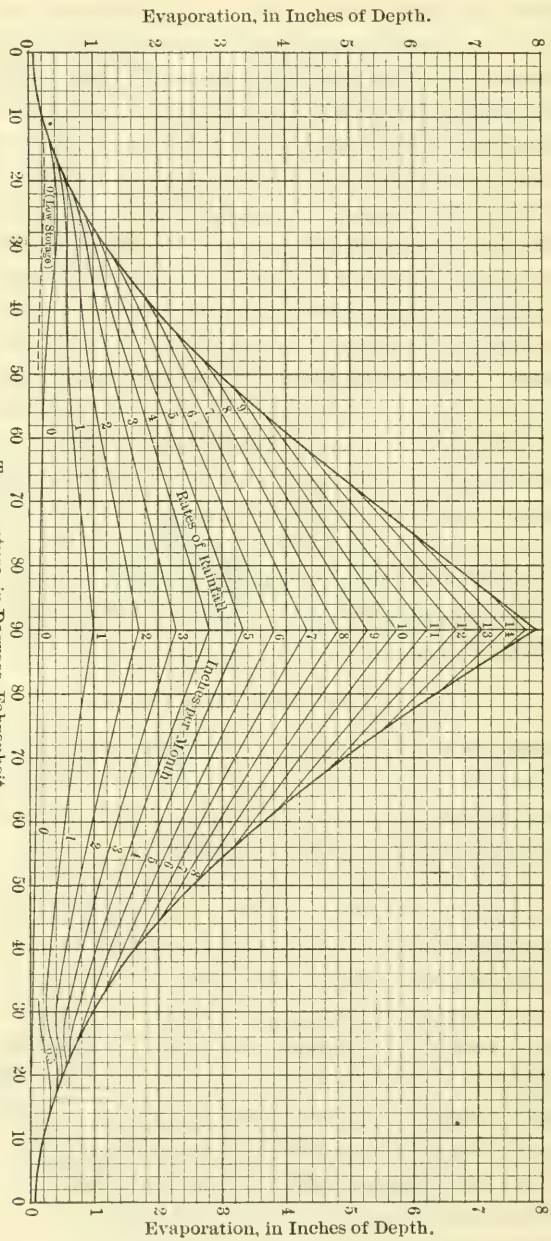
In estimating the water requirements of vegetation, the density of timber growth is not as important a factor as might at first appear. Cut-over water-sheds quickly grow up to grasses, weeds, and herbs of various kinds, which in turn are soon supplanted by shrubbery, brush, and then a growth of young trees. Areas of agricultural land not under cultivation, or after harvest and before fall plowing, soon become thoroughly covered with weeds and grasses, and hence suffer a transpiration loss perhaps fully as high as though seeded to grains and other useful vegetation. Even the rugged water-sheds of mountain ranges below the timber line are well covered with brush, grasses, moss, and other forms of low-growing vegetation. Burnt-over water-sheds with scanty covering of soil, and rock outcropping everywhere, as in the northeastern part of Minnesota, are also well covered with vegetation of one kind or another. As a consequence, the normal transpiration loss, so far as it is determined by the character of vegetation on different water-sheds, does not vary between wide limits. Except in the arid region, hardly a water-shed of considerable size can be found that is given over purely to one class of vegetation. Practically all of them are covered by mixed vegetation, including trees, shrubs, grasses, or grains. Of the streams considered, only

the three California rivers show a wide variation in transpiration losses. It is interesting to note, however, that this difference in transpiration loss does not result so much from a difference in the character of vegetation which could grow on the water-shed, so far as temperature is concerned, as a difference in the character of the vegetation which does grow on the given water-shed, as a result of differences in the quantity of water available for transpiration. For example, the grasses on the pastures of the Pit River Basin (see "Description of Water-sheds") were estimated to transpire 0.4 in. during April, 1907, when the monthly temperature was 48°, and 0.8 in. during May of the same year, when the monthly temperature was 52 degrees. The transpiration for the cultivated Root River water-shed, of Minnesota, in April, 1908, when the monthly temperature was 47° and the precipitation 2.9 in., was estimated at 0.3 in. The transpiration on the same water-shed for May, 1910, when the monthly temperature was 53° and the precipitation 2.6 in., was estimated at 0.9 in. On account of deficient precipitation, the transpiration on the Pit River water-shed during the summer, however, is limited to about 1 in., as compared with a summer transpiration on the Root River water-shed of more than 6 in.

The Evaporation Curve.

Although most of the facts on which the curve of evaporation from land areas, for various temperatures and rates of rainfall (Fig. 19), is based, have been discussed previously, the following more prominent considerations may bear further emphasis.

In the fall, when the monthly temperature reaches 20°, practically all the precipitation occurs as snow; consequently, evaporation for temperatures below 20° is no longer dependent on precipitation after the ground has been covered with snow, but entirely on temperature. Full evaporation, corresponding to the given monthly temperature, is usually possible throughout the winter. After the temperature rises above 20°, in spring, the evaporation again depends largely on available moisture, as determined mainly by precipitation. Nevertheless, a considerable constant evaporation is still possible, irrespective of precipitation, because a certain quantity of snow and ice is almost always present on the ground while the monthly temperature ranges from 20 to 35 degrees. After the snow has disappeared,



EVAPORATION FROM LAND AREAS FOR VARIOUS TEMPERATURES AND RATES OF RAINFALL.

FIG. 19.

there will still be a relatively large constant evaporation, irrespective of the rainfall, unless the winter precipitation has been distinctly deficient.

A gradual reduction in the constant evaporation has been assumed for the summer. It is realized, of course, that the constant evaporation during the summer depends, in a measure, on the rainfall of each previous month. In making detailed computations of evaporation losses, the constant evaporation is readily varied by one or two tenths of an inch, in accordance with apparent variations in storage. On some water-sheds, when the fall precipitation is very low and the temperature remains above 30°, the right-hand portion of the curve is used for January and sometimes also for February, that is, when the storage is practically exhausted and there is no snow on the ground, the constant of evaporation otherwise used practically vanishes and the evaporation is entirely proportional to the rainfall. In the same way, when the fall rains are copious and the ground-water supply is abundant, a constant of evaporation one or two tenths higher than that given by the curve may be used to advantage.

The portions of the limiting curve below temperatures of approximately 35° represent evaporation from snow and ice surfaces. At the higher temperatures the limiting curve represents values somewhat less than the evaporation from shallow water. The quantity evaporated out of each inch of rainfall becomes less and less as the monthly precipitation increases, varying more rapidly at the lower than at the higher rates of precipitation.

To the values of evaporation, in inches of depth per month, as taken off the curve, a coefficient must be applied to reduce these quantities to actual evaporation from the given water-shed. This coefficient ranges from about 0.95 to 1.25 for most water-sheds of the Northwest, and for similar ones elsewhere. The coefficient to be used depends on topography, vegetal cover, soil, subsoil, humidity, and wind. An extremely high coefficient of evaporation would result from flat topography devoid of vegetation, moderately pervious, shallow soil underlain with impervious subsoil or rock, low humidity, and high wind velocity. An extremely low coefficient would result from rugged topography, bare scanty soil underlain with rock, high humidity, and low wind velocity. Between these extremes the usual working values will be found. With a little experience, one can select coefficients

for different water-sheds with considerable accuracy. On the fifteen streams cited later, for example, the writer's first choice of coefficients never varied from the final choice by more than one-tenth and seldom by more than one-half a tenth. The final coefficient was adopted after the computed precipitation minus losses for 3 or 4 years had been checked against the observed run-off.

The Transpiration Curve.

The base values for total transpiration, in inches of depth, during the growing season on any given water-shed, are selected with reference to the character of the vegetation and the length of the growing season on that water-shed, giving consideration also to available sunshine. In the following computations a normal seasonal transpiration of about 9 in. has been assumed for small grains, grasses, and other agricultural crops, 8 in. for deciduous trees, 4 in. for evergreen trees, and 6 in. for small trees and brush. The normal monthly distribution of this total seasonal transpiration is based mainly on temperature. To obtain actual transpiration in any given month, however, the values taken from the transpiration curve, after being multiplied by a coefficient, must be further modified on the basis of available moisture. Where precipitation minus evaporation for a given month is insufficient to meet the normal plant requirements for that month, the ground-water is drawn on to a varying extent, depending on the character of the root system of the given vegetation, the depth and character of the soil, and the quantity of surface soil storage, as determined by the precipitation minus losses for previous months.

Computing Annual Run-Off.

The main features of the writer's method of computing run-off to supplement observed stream-flow data may be summarized as follows:

I.—Collection of physical data.

A.—Rainfall and temperature data for stations on and near the given water-sheds from which monthly rainfall and temperature for the water-shed are estimated. Rates of excessive rainfall at different seasons of the year, as indicated by Weather Bureau observations at the nearest regular station. In case rainfall and temperature data are meager,

charts showing isotherms and isohyetals for the portion of the State in which the water-shed is situated are of assistance.

B.—Data relating to wind velocity, relative humidity, and any other prominent weather characteristics.

C.—Data relating to topography, vegetal cover, soil and subsoil, as affecting evaporation.

D.—Data relating to character and density of vegetation and length of growing season, with reference to temperature and hours of sunshine.

E.—Data relating to area of open water surfaces, swamps, and marshes.

II.—Determination of losses.

A.—Evaporation from water area.

1.—Monthly evaporation corresponding to given temperature and season, taken off curve, Fig. 12, and multiplied by percentage of water surface, based on data under *I-E,* and coefficient based on data under *I-B.*

B.—Evaporation from land area.

1.—Determination of coefficient for given water-shed, based principally on physical data under *I-C* and *I-B.*

2.—Determination of evaporation, in inches depth per month, corresponding to given monthly temperature and rainfall for given season of year, from curve of evaporation from land areas, Fig. 19, and multiplication of the same by percentage of land area and coefficient determined under *II-B-1.*

C.—Transpiration from land area.

1.—Determination of normal seasonal transpiration, based on physical data under *I-D.*

2.—Determination of transpiration coefficient by finding ratio between seasonal transpiration determined from base curve of transpiration (Fig. 17) for the normal monthly temperatures for the given water-shed, and the normal seasonal transpiration determined under *II-C-1.*

3.—Determination of monthly transpiration by applying transpiration coefficient to monthly values taken off transpiration curve for given monthly temperatures, and modification of these monthly values on basis of rainfall, percolation, and storage.

III.—Determination of total loss by summation of monthly losses from land and water areas, the deduction of these monthly losses from the monthly precipitation, and summation of these monthly residuals to give the annual yield of the given water-shed, with or without correction of this annual total for fall surface run-off, or changes in ground and surface storage.

IV.—Where the annual yield and its distribution throughout the year are both desired, additional curves similar to those for the Root River water-shed, and computations similar to those given in Table 35, for the same water-shed, must be made. When the more detailed computations, as here indicated, are carried out, it is possible to make more accurate estimates of transpiration during months of deficient rainfall, because more accurate values of soil and subsoil storage are available.

Characteristic Water-sheds.

In order to show the practical application of the writer's method of computing run-off, water-sheds of widely different characteristics have been selected, varying in location from Virginia to California, and from Minnesota to Texas.

Table 5 gives the coefficients of evaporation from land areas for each of the fifteen water-sheds considered, together with the principal observed and computed physical data pertaining to each.

Description of Water-sheds.

In order to show the relation between evaporation coefficients, transpiration, and water-shed characteristics, a brief description of the fifteen water-sheds, giving the facts on which the choice of coefficients is mainly based, follows. It is realized that the information given is meager, and to those intimately familiar with the various water-sheds, perhaps not entirely accurate. Whatever discrepancies exist, however, would indicate that reasonably accurate computed values

of run-off can be secured by the writer's method, notwithstanding this fact. When reasonably good rainfall and temperature data are available, considerable latitude in the choice of coefficients is allowable.

TABLE 5.—OBSERVED AND COMPUTED PHYSICAL DATA FOR FIFTEEN WATER-SHEDS.

Name of water-shed.	Years of record.	Evaporation coefficient.	Area, in square miles.	OBSERVED AND COMPUTED PHYSICAL DATA—MEAN ANNUAL.							
				Rainfall.	Temperature.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Computed run off.	Observed run-off.
Mississippi	17	1.20	19 500	27.3	41	14.4	7.7	22.1	5.2	5.23	5.31
Little Fork	5	1.10	1 720	23.9	37	11.2	6.9	18.1	5.8	5.80	....
										*5.15	*5.13
Minnesota	5	1.25	6 300	22.7	43	14.1	7.5	21.6	1.1	1.1	....
									*0.77	*0.77	*0.70
Root	6	1.225	1 560	31.2	45	16.5	8.6	25.1	6.1	6.10	....
										*5.40	*5.20
Ottertail	6	1.10	1 310	23.0	40	13.5	6.6	20.1	2.9	2.80	....
										†2.66	†2.59
St. Croix	11	1.05	5 930	30.0	41	13.1	7.0	20.1	9.9	9.90	9.60
Ohio	14	0.875	23 820	41.1	51	14.8	5.8	20.6	20.5	20.50	22.00
Tohickon Creek	24	0.90	102	48.9	51	16.7	7.0	23.7	25.2	25.2	26.10
James	7	0.925	6 230	42.1	54	16.3	7.0	23.3	18.8	18.90	18.00
Roanoke	9	0.90	390	42.6	57	16.9	7.0	23.9	18.7	18.60	17.70
Tombigbee	9	1.05	4 440	49.2	62	22.8	8.4	31.2	18.0	18.00	17.10
Colorado	10	1.20	37 000	26.9	66	17.7	8.1	25.8	1.1	1.06	0.74
Sacramento	9	0.85	10 400	32.2	52	8.5	2.4	10.9	21.3	21.3	20.40
Pit	6	1.10	2 950	14.8	48	6.9	3.0	9.9	4.9	4.9	....
									*3.87	*3.87	*3.92
McCloud	6	0.60	608	61.9	55	8.2	2.4	10.6	51.3	51.3	54.00

* Four years' records.

† Five years' records.

In the State of Minnesota.

Mississippi River Water-shed, at Minneapolis.—Practically the entire water-shed is heavily covered with glacial drift, and considerable portions are distinctly sandy. Elevations along the river vary from about 800 to 1 475 ft., making the average fall a little less than 1½ ft. per mile.

Although considerable lake and swamp area is known to exist, particularly in the northern part of the State, no allowance has been made for loss from water surface in the computations, except that the coefficient of evaporation has been increased slightly.

Only a comparatively small portion of the water-shed in the northern part of the State is still covered with dense forests. Much of the area has been cut over and is now grown up to birch, poplar,

and other brush; there is a scattering growth of jack and other pines. Perhaps one-third of the area may be considered under cultivation.

Little Fork River Water-shed, at Little Fork, Koochiching County.—This water-shed lies in the extreme northern part of the State, and is characterized by distinctly flat topography, with only a few hills rising more than 50 or 75 ft. above the plain. The general slope is insufficient to provide good drainage, consequently there are extensive areas of swamp.

The soil is quite clayey. Very little land is under cultivation. Although logging has been carried on extensively for some years, the cut-over area has quickly grown up with poplar, birch, jack and other pines, so that the water-shed, in general, must be considered as densely wooded.

Elevations in the basin range from about 1 100 to 1 400 ft., except for a narrow strip at the head-waters in the Iron Ranges, where elevations run up to 1 700 ft. and higher. The slope along the main stream averages about 2 ft. per mile.

Minnesota River Water-shed, at Montevideo, Chippewa County.—This water-shed lies in the western part of the State, and a very large portion of it is flat open prairie. Almost all of it is under cultivation. Near the head-waters there are large areas of marsh on clayey soil that contribute little run-off. Although the main stream lies in a valley which is from 100 to 200 ft. below the general level of the water-shed, most of the tributary streams run in comparatively shallow valleys, except for V-shaped gullies which they have cut through the bordering bluffs as they approach the main stream. There are ridges with steep slopes over a portion of the southern and south-western side of the water-shed. The slope of the river in the reaches above Big Stone Lake is about 20 ft. per mile. Here the bed is frequently dry. From Big Stone Lake to the Montevideo Station the slope averages only about 0.6 ft. per mile.

Transpiration is limited mainly by the quantity of available water, there being a deficiency during almost every growing season. Transpiration is assumed over the entire water-shed, even though 5% is estimated to be lake and open marsh, because of the fact that the excessive transpiration of the long marsh grasses more than compensates for the lack of transpiration over lake areas.

The run-off from this water-shed is so extremely small that it was believed desirable to take into consideration losses from water areas. The exact area of lake and open marsh was not known, but was estimated to be about 5 per cent.

Root River Water-shed, at Houston, Houston County.—This water-shed lies in the southeastern corner of the State. The topography varies from relatively flat to gently undulating. The streams, however, almost all run in deep V-shaped valleys, that of the main stream being several hundred feet below the general level of the surrounding country.

Most of the land is under cultivation. Timbered areas are practically confined to the watercourses.

The soil is largely clayey loam, rather impervious, subjecting the precipitation to high evaporation losses. The slope of the water-table to the deep drainage courses is steep, even though the water-table lies far below the surface of the ground.

The average slope along the Root River is about 6 ft. per mile.

Ottertail River Water-shed, at Fergus Falls, Ottertail County.—The entire water-shed lying to the west of the center of the State, and just north of that of the Minnesota River, previously described, is dotted with lakes of various sizes, and many have no outlets. Others have outlets only in high water. It is estimated that the water surfaces aggregate 15% of the entire area.

The topography is prominently rolling, morainic, and knolly, a portion at the southern extremity being more gently undulating. The entire area is heavily covered with drift, varying from clays to sand, gravel, and stony moraines.

The water-shed, on the whole, is lightly timbered, with considerable land under cultivation in the southern portion. Good natural storage is manifested by well-sustained stream flow during the winter and during dry seasons.

In the States of Minnesota and Wisconsin.

St. Croix River Water-shed, at St. Croix Falls, Polk County, Wis.—This water-shed lies on both sides of the eastern boundary and nearly opposite the center of the State. The topography, in general, is undulating. Practically the entire basin is thickly covered with glacial drift, much of it sand and gravel. There are no rock outcrops,

except near the streams. The northern portion is distinctly rolling. Part of the southern portion is quite flat. In the southeast corner there is an area consisting of knolls and basins.

There are virtually no marshes on the water-shed. In the Wisconsin portion, however, the number of lakes is considerable and many of them have no outlet. Good natural ground storage is shown by the stream flow.

Elevations along the river vary from 750 to 1 000 ft., the average slope of the stream being 2½ ft. per mile.

In the State of West Virginia.

Ohio River Water-shed, at Wheeling.—The Ohio River is formed by the junction of the Allegheny and the Monongahela, at Pittsburgh, Pa., about 90 miles above Wheeling.

The Allegheny River Basin constitutes about two-thirds of the entire area, and the Monongahela the remaining third. The northeastern portion is rolling and hilly. Practically the entire remaining portion of both the Allegheny and the Monongahela water-sheds are mountainous. The depth of soil, on the whole, is small. The slope of the Allegheny is about 6½ ft. per mile.

Winter conditions in the upper part of the Allegheny Basin are severe, snowfall being heavy and ice forming to considerable thickness.

The Monongahela rises in West Virginia, and flows in a generally northerly direction to its junction with the Allegheny at Pittsburgh.

The upper portion of the water-shed is mountainous; the slopes of the valleys are steep, and in many places precipitous. The lower portion of the river valley, though less mountainous, is still very rolling, and the soil is quite rocky, with but little power of absorption.

Most of the water-shed of the Ohio above Wheeling consists of cut-over land, which formerly was heavily forested, largely with hardwood timber.

Ice conditions are not very severe, due to the southern latitude, and the snowfall, except in the mountains, is comparatively light.

In relative humidity, the Ohio River water-shed, above Wheeling, and the Mississippi, above Minneapolis, are approximately the same, but the average wind velocity in the Ohio Basin is very much less than that in the Mississippi.

In the State of Pennsylvania.

Tohickon Creek Water-shed, at Point Pleasant.—This water-shed is in Bucks County, north of Philadelphia. The Creek flows eastward into the Delaware River.

According to Hering, in 1885, 76 sq. miles were cultivated and improved land, and 26 sq. miles were untillable and wooded.

The fall of the stream is about 20 ft. per mile. The slopes in the basin are very steep.

In the State of Virginia.

James River Water-shed, at Cartersville.—The James River rises in the Allegheny Mountains, and the Station of Cartersville lies on a plateau about 50 miles above Richmond.

The portion of the water-shed in the Allegheny Mountains is very broken and has steep slopes. On the Piedmont Plateau the topography is rolling, and the uplands are rounded, with consequently a small range in elevation. Elevations in the head-waters run up to 4 000 ft.

The mountain sides are forested, but a portion of the lower water-shed is under cultivation.

Roanoke River Water-shed, at Roanoke.—The Roanoke River rises among the eastern foot-hills of the Appalachian Mountains, and flows southeastward to Albemarle Sound, in North Carolina.

The topography above Roanoke is rugged, the soil cover scant, and little of the land is under cultivation.

The range of altitude above Roanoke is about 2 000 ft. The slopes along the stream are steep.

In the State of Mississippi.

Tombigbee River Water-shed, at Columbus.—The country, on the whole, is flat. The streams have a gentle uniform slope. Hardly one-quarter of the water-shed has been improved. The remainder is still heavily forested. The soil is deep and, for the most part, is a rather heavy loam. The humidity is not much higher than that of the Northwest. An average of about 4 in. of snow falls annually on this water-shed.

In the State of Texas.

Colorado River Water-shed, at Austin.—This water-shed is comparatively flat, but the slopes are rather pronounced. Timber is found

along the watercourses and at the higher elevations only. The soil is deep, and not especially sandy.

The humidity and wind velocity both differ but slightly from those prevailing throughout most of the Northwest.

In the State of California.

Sacramento River Water-shed, at Red Bluff, Including its Tributaries, the Pit and the McCloud.—The Sacramento River drains the basin between the Coast and Trinity Ranges on the west, and the Sierra Nevadas on the east. The western slope constitutes a very narrow belt. The main portion of the discharge is derived from the Pit River Basin, which has its head-waters in the Warner Mountains in the extreme northeastern part of the State.

Perhaps half of this water-shed is covered with timber. The other half supports a fair growth of grass, and is used extensively for pasturage. The soil covering the underlying lava is quite scant. Most of the precipitation, particularly over the head-waters of the stream, is in the form of snow.

The Pit River Basin comprises that portion of the Sacramento River water-shed lying to the northeast. It is relatively flat and with little timber growth, most of it being pasture land. Most of the area above Bieber lies at an elevation of from 4 000 to 5 000 ft. above sea level.

The McCloud River enters the Pit from the north a short distance above the place where the latter discharges into the Sacramento. It drains the southern slope of Mt. Shasta. Most of the water-shed is at an elevation ranging between 3 000 and 5 000 ft. The slopes are very steep and practically all the minor tributaries rise among mountain peaks. A very large proportion of the precipitation is snow, which gradually melts and feeds the stream during the summer when the precipitation is practically nil.

It was impossible to obtain either temperature or precipitation records directly on the McCloud River water-shed. For portions of the time, data were available for the Stations of Sisson, Dunsmuir, Delta, Shasta, and Redding, on the Sacramento just west of the McCloud. The precipitation data for these stations are believed to give approximately correct values for the McCloud River water-shed. The temperatures, based on the observed data at these stations, however, are believed to give too high values.

Application of Method to Characteristic Water-sheds.

It is believed that the diagrams, Figs. 20 to 29, for ten of the fifteen water-sheds to which the method has been applied, indicate clearly the inter-relationship between temperature, monthly distribution of rainfall, and run-off, irrespective of differences in the physical characteristics of the several water-sheds.

Three distinct types of monthly rainfall distribution are shown. The first is characterized by a practically uniform distribution of rainfall throughout the year, as exemplified by Tohickon Creek in particular. A somewhat similar distribution of rainfall prevails on the water-sheds of the Ohio, the James, the Roanoke, and the Tombigbee Rivers.

With the exception of the Colorado and the Tombigbee, the mean summer temperatures do not differ greatly on the several water-sheds shown. The mean winter temperatures, however, differ quite widely, varying from about 10° on Minnesota water-sheds to about 30° on those in Pennsylvania, Virginia, West Virginia, and California, and nearly 50° for the Colorado River water-shed in Texas and the Tombigbee in Mississippi.

In the second type of rainfall distribution, the precipitation is high during the summer, and low during the winter, as exemplified by the St. Croix, the Mississippi, and the Colorado River water-sheds.

This type of rainfall distribution results in proportionately large evaporation losses, in both summer and winter. Although the winter evaporation losses average only 0.2 or 0.3 in. per month in Minnesota, they are proportionately large because the precipitation averages less than 1 in. per month during this period. The high summer precipitation results in heavy evaporation and transpiration losses, on account of high temperatures.

It is interesting to note that if the summer precipitation of about 4 in. per month on the St. Croix and Mississippi River water-sheds occurred at temperatures prevailing on the Colorado River water-shed, the losses out of precipitation due to temperature alone would reduce the run-off from these water-sheds to approximately the same as that observed on the Colorado.

Similarly, if the winter precipitation of about 1 in. per month occurred at the Texas winter temperatures of nearly 50°, instead of at

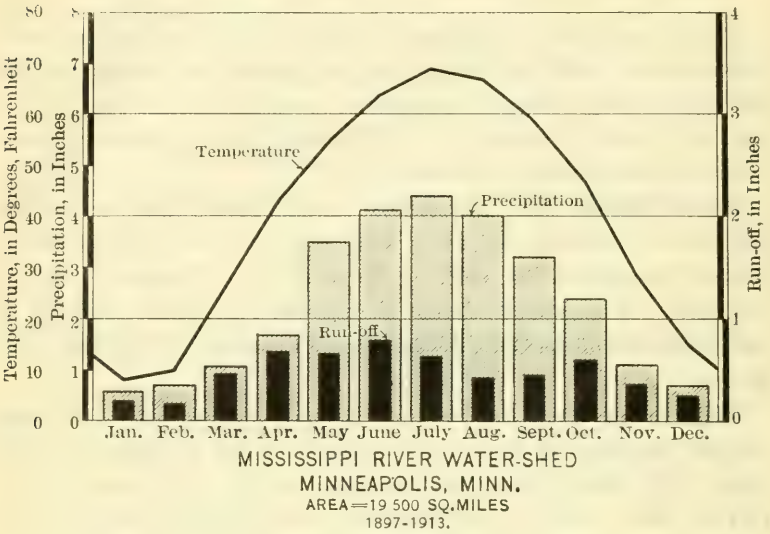

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Temperature
Precipitation
Run-off
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
MISSISSIPPI RIVER WATER-SHED
MINNEAPOLIS, MINN.
AREA=19 500 SQ.MILES
1897-1913.

FIG. 20.

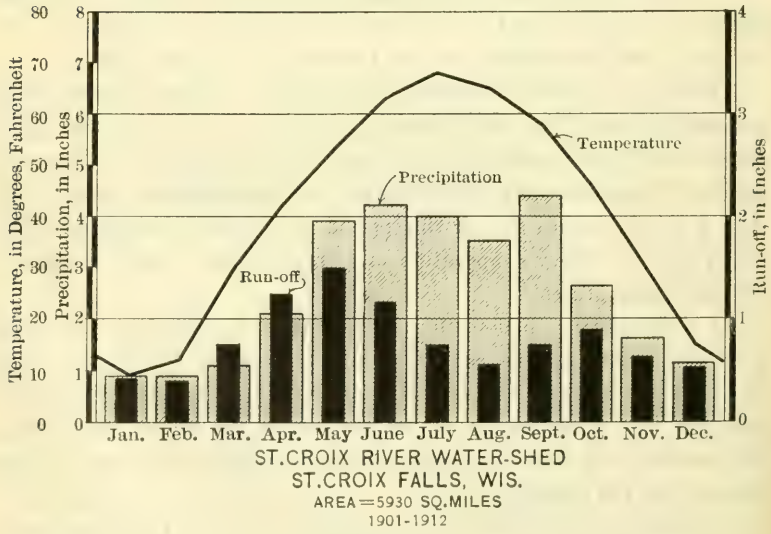

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Temperature
Precipitation
Run-off
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
ST.CROIX RIVER WATER-SHED
ST.CROIX FALLS, WIS.
AREA=5930 SQ.MILES
1901-1912

FIG. 21.

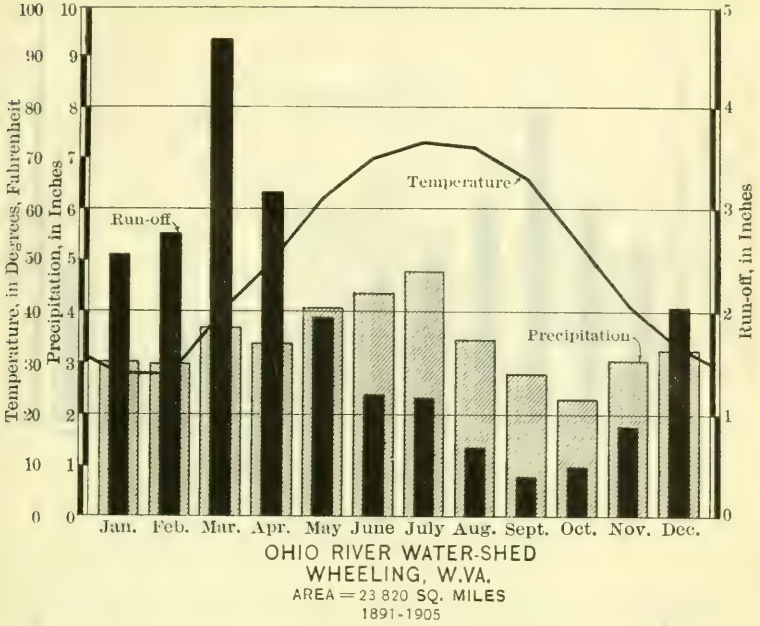

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Run-off
Temperature
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
OHIO RIVER WATER-SHED
WHEELING, W.VA.
AREA = 23 820 SQ. MILES
1891-1905

FIG. 22.

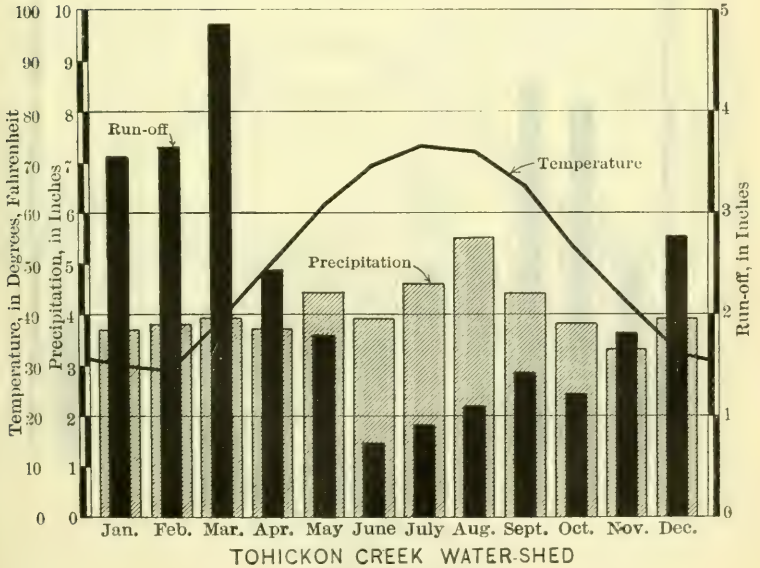

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Run-off
Temperature
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
TOHICKON CREEK WATER-SHED
POINT PLEASANT, PA.
AREA = 102 SQ. MILES
1887-1911

FIG. 23.

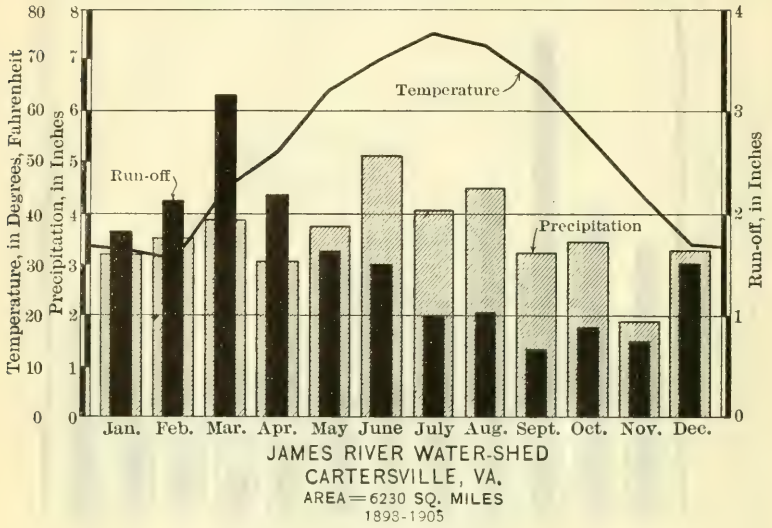

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Temperature
Run-off
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
JAMES RIVER WATER-SHED
CARTERSVILLE, VA.
AREA = 6230 SQ. MILES
1898-1905

FIG. 24.

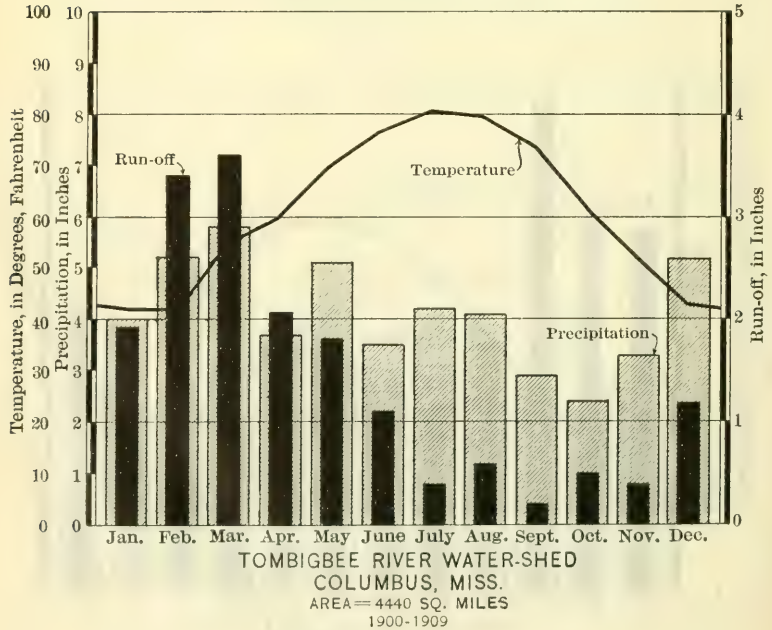

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Run-off
Temperature
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
TOMBIGBEE RIVER WATER-SHED
COLUMBUS, MISS.
AREA = 4440 SQ. MILES
1900-1909

FIG. 25.

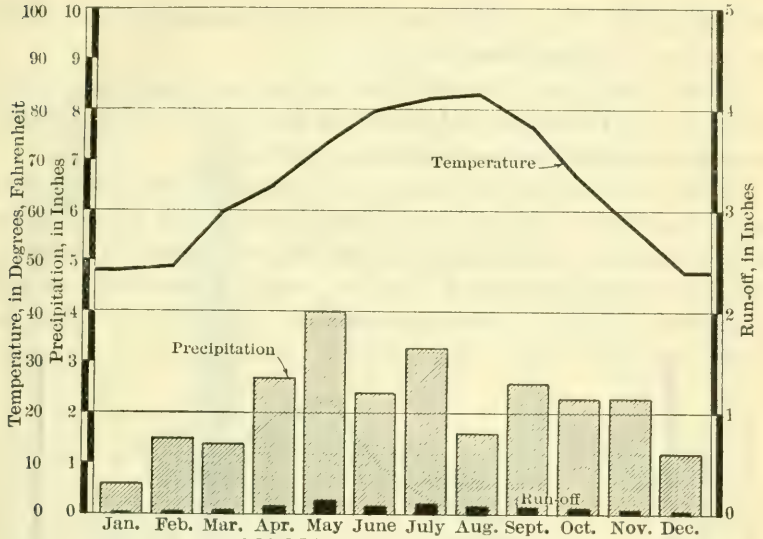

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Temperature
Precipitation
Run-off
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
COLORADO RIVER WATER-SHED
AUSTIN, TEXAS
AREA = 37 000 SQ. MILES
1901-1910

FIG. 26.

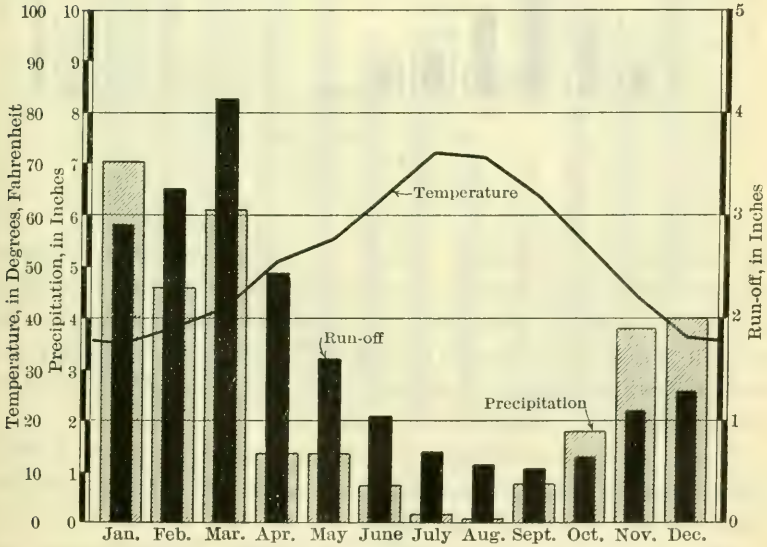

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Temperature
Run-off
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
SACRAMENTO RIVER WATER-SHED
RED BLUFF, CAL.
AREA = 10 400 SQ. MILES
1902-1911

FIG. 27.

the Minnesota winter temperatures of 10°, the evaporation losses would be increased at least threefold.

In the third type of rainfall distribution, very high precipitation occurs during the winter, and practically none at all during the summer, as exemplified by the California streams.

At the temperatures prevailing on the Sacramento River water-shed above Red Bluff, a winter precipitation of about 5 or 6 in. per month permits maximum evaporation, so that all precipitation in excess of this goes to swell the run-off. As the monthly evaporation at winter temperature ranges in the neighborhood of 1 in. to 1½ in., it must be

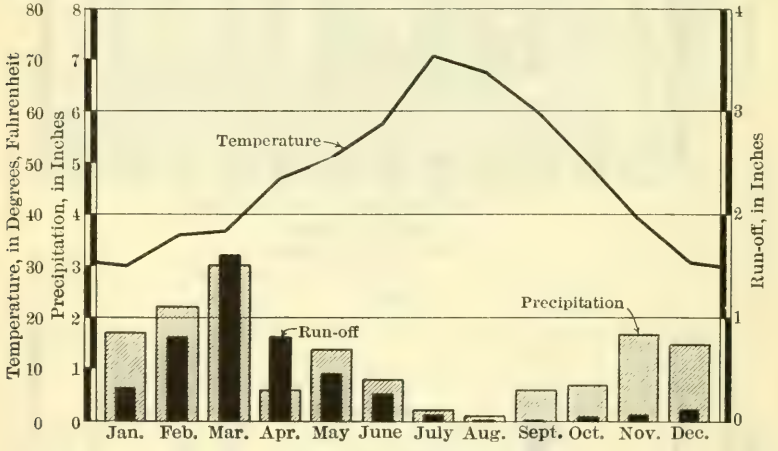


FIG. 28.

apparent that a precipitation of 5, 6, or 7 in. per month is certain to produce very heavy run-off. As practically no precipitation falls during the summer, vegetation is dependent almost entirely on the rainfall stored in the ground during the previous rainy season; and stream flow, likewise, is dependent on ground storage and surface storage in lakes or as snow on the mountains.

For a given temperature distribution, evaporation losses will be proportionately least for the last type of rainfall distribution, greatest for the second type, and intermediate for the first type.

The necessity of taking into account monthly temperatures in any attempt at computing run-off, may bear emphasis. The mean tem-

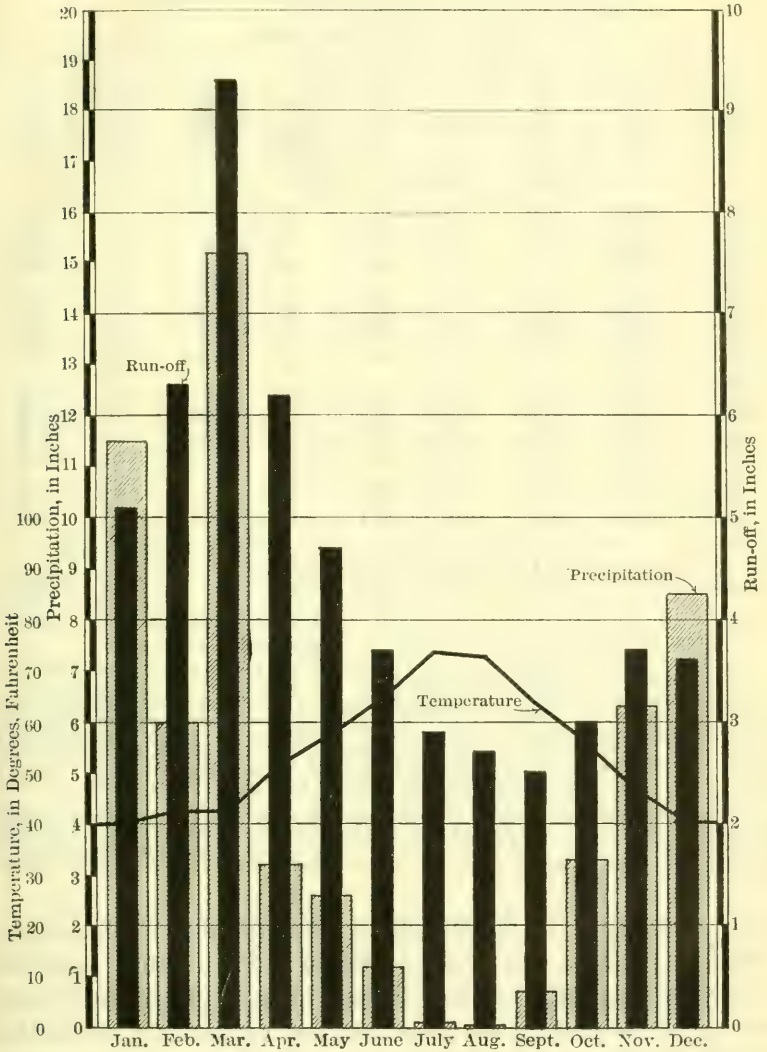

Run-off
Precipitation
Temperature
Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
McCLOUD RIVER WATER-SHED
GREGORY, CALIFORNIA
AREA=608 SQ.MILES
1902-1908

FIG. 29.

TABLE 6.—SUMMARY OF DATA AND COMPUTATIONS FOR THE MISSISSIPPI RIVER WATER-SHED, ABOVE MINNEAPOLIS, MINN.

Area = 19 500 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Surface run-off.	Computed run-off.	Observed run-off.†
1897	32.58	15.6	8.8	24.4	8.2	0	8.2	8.16
1898	24.29	13.4	7.5	20.9	3.4	0	3.4	3.85
1899	32.54	16.3	8.8	25.1	7.4	0	7.4	6.64
1900	28.14	16.2	8.4	24.6	3.5	0	3.5	3.82
1901	22.57	12.4	6.8	19.2	3.4	0	3.4	3.31
1902	25.80	14.7	7.8	22.5	3.3	0.2	3.5	3.30
1903	32.85	16.3	8.5	24.8	8.0	—0.2	7.8	6.41
1904	26.59	13.4	7.8	21.2	5.4	0	5.4	5.45
1905	32.68	16.7	7.8	24.5	8.2	0.6	8.8	9.71
1906	32.48	16.1	7.9	24.0	8.5	—0.2	8.3	9.28
1907	24.46	12.0	6.0	18.0	6.5	—0.2	6.3	6.68
1908	27.60	14.8	7.2	22.0	5.6	0	5.6	6.22
1909	24.04	13.1	6.7	19.8	4.2	0.1	4.3	5.23
1910	17.96	9.4	6.0	15.4	2.6	—0.3	2.3	2.33
1911	27.54	15.5	8.5	24.0	3.5	0	3 5	2.20
1912	23.01	13.0	7.0	20.0	3.0	0	3.0	3.52
1913	28.83	15.9	8.5	24.4	4.4	0	4.4	4.23
Total..	463.96	244.8	130.0	374.8	89.1	.....	89.0	90.34
Mean..	27.3	14.4	7.7	22.1	5.2	.....	5.2	5.31

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

NOTE.—A run-off year, beginning March 1st and ending on the last of February following, is assumed to correspond with the rainfall year beginning November 1st and ending on the following October 31st. It is assumed that practically all excess ground-water available for run-off on November 1st of any year will have appeared as run-off before the following March 1st. Where the temperature for November was high, an estimate has been made of the precipitation which appeared immediately as surface run-off, instead of being held through the winter as snow and added to the run-off of the following year.

It may be noted that during 1911 the discharge of most of the Minnesota streams was exceptionally low, even though the rainfall was above the average. This can be accounted for by the fact that a large portion of the precipitation was used to replenish excessive depletion of ground storage during 1910, which was the dryest year on record for Minnesota.

perature for April, on the Mississippi River water-shed, for example, varied in 17 years from 32.2° in 1907 to 48.8° in 1910. Assuming a precipitation of 3 in. per month, this change in temperature would result in a variation in the evaporation loss of about 0.4 in., besides a loss due to transpiration of 0.2 or 0.3 in., making a total possible difference in run-off of from 0.6 to 0.7 in., resulting from the observed temperature variation for this one month alone. An equal difference in evaporation loss is indicated by the variation in May temperatures from 44.9° in 1907 to 60.3° in 1911. This variation in temperature

TABLE 7.—DATA AND COMPUTATIONS FOR THE MISSISSIPPI RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches			
				From curve.	Actual.		
1901.							
Nov.....	28.5	0.97	.....	0.20	0.24	0.24	0.73
Dec.....	13.8	0.55	.....	0.30	0.36	0.36	0.19
1902.							
Jan.....	14.8	0.37	.....	0.35	0.42	0.42	—0.05
Feb.....	15.6	0.50	.....	0.35	0.42	0.42	0.08
Mar.....	33.6	0.72	.....	0.50	0.60	0.60	0.12
April...	41.7	1.45	0.1	0.75	0.90	1.00	0.45
May....	57.1	5.32	1.3	2.25	2.70	4.00	1.32
June....	62.1	3.28	1.6	1.70	2.04	3.64	—0.36
July....	69.9	3.96	2.0	2.20	2.64	4.64	—0.68
Aug....	65.2	4.76	1.7	2.12	2.54	4.24	0.52
Sept	55.1	1.69	0.8	0.75	0.90	1.70	—0.01
Oct.....	46.7	2.23	0.3	0.75	0.90	1.20	1.03
	.....	25.80	7.8	12.22	14.66	22.46	3.34
1906.							
Nov.....	31.0	2.15	.....	0.45	0.54	0.54	1.61
Dec....	14.9	0.93	.....	0.30	0.36	0.36	0.57
1907.							
Jan.....	1.5	1.22	.....	0.08	0.10	0.10	1.12
Feb.....	14.3	0.53	.....	0.30	0.36	0.36	0.17
Mar.....	26.5	0.92	.....	0.50	0.60	0.60	0.32
April...	32.2	0.80	.....	0.50	0.60	0.60	0.20
May....	44.9	2.82	0.3	*1.00	1.20	1.50	1.32
June....	63.9	3.23	1.6	*1.60	1.92	3.52	—0.29
July....	68.7	2.63	1.6	*1.45	1.74	3.34	—0.71
Aug....	66.5	4.14	1.5	*1.90	2.28	3.78	0.36
Sept....	56.2	3.83	0.9	1.50	1.80	2.70	1.13
Oct.....	45.4	1.26	0.1	0.45	0.54	0.64	0.62
	.....	24.46	6.0	10.03	12.04	18.04	6.42

* Low storage.

NOTE.—The column for actual evaporation represents values in the preceding column multiplied by an evaporation coefficient for the given water-shed and a percentage of the land area.

would also result in a variation in transpiration loss of at least 1 in., or a total difference in annual run-off of about 25%, resulting from the observed variation in temperature for this one month alone.

Similar variations occur in monthly summer temperatures. August, 1903, for example, had a temperature of 62.7° as compared with 74.1°

TABLE 8.—Summary of Data and Computations for the Little Fork River Water-shed, Above Little Fork, Minn.

Area = 1 720 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Surface run-off.	Computed run-off.	Observed run-off.†
1909.....	28.4	12.8	7.2	20.0	8.4	0.0	8.4	.....
1910.....	19.0	8.9	6.3	15.2	3.8	0.0	3.8	3.75‡
1911.....	23.4	11.5	7.2	18.7	4.7	0.0	4.7	4.62
1912.....	22.0	10.8	6.9	17.7	4.3	0.0	4.3	4.21
1913.....	26.9	12.3	6.8	19.1	7.8	0.0	7.8	7.95
Total.....	119.7	56.3	34.4	90.7	29.0	.....	29.0	20.53
Mean.....	23.9	11.2	6.9	18.1	5.8	.....	5.8§(5.15)	5.13

* From November 1st of previous year to October 31st of given year.

† From April 1st of given year to March 31st of following year.

‡ Part estimated.

§ Mean of years over which observations extend.

Note.—No allowance for changes in storage has been made on this water-shed because the period from the close of the rainfall year to the beginning of the run-off year is 1 month (20% longer than that for the other Minnesota water-sheds considered).

The topography is quite flat, with practically no lake areas, and the soil clayey, so that little capacity for storage is to be expected. This is borne out by the available stream-flow records, which show comparatively little variation in the late winter discharge.

These records also show practically no surface run-off during November.

for August, 1900. Assuming a precipitation of 5 in. per month, the evaporation loss alone, as given by the curve, would vary by 0.8 in. Applying a coefficient of 1.2 would make the variation in the evaporation loss practically 1 in.

Similar changes in temperature occur on other water-sheds. On the Tohickon Creek, for example, within a period of 24 years, the January temperature varied from 19° in 1893 to 38° in 1890. The February temperature varied from 22° in 1895 to 40° in 1909, and the August temperature from 68° in 1889 to 76° in 1900. All these differences in temperature, if not taken into account in computing losses out of rainfall, would make a substantial difference in the computed run-off.

Tables 6 to 34 contain data and computations for fifteen water-sheds, together with detailed computations for characteristic years on each.

TABLE 9.—DATA AND COMPUTATIONS FOR THE LITTLE FORK RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1908.							
Nov.....	29	0.8	.....	0.20	0.22	0.22	0.58
Dec.....	11	0.8	.....	0.20	0.22	0.22	0.58
1909.							
Jan.....	2	1.6	.....	0.07	0.08	0.08	1.52
Feb.....	6	0.6	.....	0.12	0.13	0.13	0.47
Mar.....	22	0.8	.....	0.50	0.55	0.55	0.25
Apr.....	30	1.0	.....	0.60	0.66	0.66	0.34
May.....	49	2.6	0.9	1.15	1.27	2.17	0.43
June.....	63	1.9	1.6	1.15	1.27	2.87	—0.97
July.....	67	4.3	1.7	2.25	2.48	4.18	0.12
Aug.....	66	7.0	1.7	3.00	3.30	5.00	2.00
Sept.....	56	3.4	1.1	1.40	1.54	2.64	0.76
Oct.....	42	3.6	0.2	1.00	1.10	1.30	2.30
	.....	28.4	7.2	11.64	12.82	20.02	8.38
Nov.....	29	1.6	.....	0.32	0.35	0.35	1.25
Dec.....	8	1.4	.....	0.15	0.16	0.16	1.24
1910.							
Jan.....	8	0.4	.....	0.17	0.19	0.19	0.21
Feb.....	3	0.5	.....	0.08	0.09	0.09	0.41
Mar.....	37	0.4	.....	0.45	0.50	0.50	—0.10
Apr.....	45	2.4	0.2	1.00	1.10	1.30	1.10
May.....	49	1.3	0.8	0.70	0.77	1.57	—0.27
June.....	67	0.9	1.3	0.70	0.77	2.07	—1.17
July.....	67	3.5	1.6	1.90	2.09	3.69	—0.19
Aug.....	60	1.3	1.1	0.65	0.72	1.82	—0.52
Sept.....	55	3.0	1.1	1.20	1.32	2.42	0.58
Oct.....	46	2.3	0.2	0.80	0.88	1.08	1.22
	.....	19.0	6.3	8.12	8.94	15.24	3.76

MONTHLY DISTRIBUTION OF RUN-OFF.

There is such an intimate relationship between the run-off and the physical characteristics of a water-shed, the daily temperature, and the character and rate of precipitation, that only the most detailed analysis can possibly result in even a reasonably accurate estimate of the daily and monthly distribution of the annual run-off.

In comparing daily or monthly computed stream flow with observed data, it is well to keep in mind, however, that both observed and com-

TABLE 10.—Summary of Data and Computations for the Minnesota River Water-shed, Above Montevideo, Minn.

Area = 6 300 sq. miles.

Year.	Rainfall.*	Evaporation.		Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.†
		Water.	Land.				
1909	23.72	1.49	12.75	7.0	21.24	2.48	‡
1910	17.76	1.78	8.84	6.5	17.12	0.64	0.90
1911	24.83	1.63	14.00	8.3	23.93	0.90	0.27
1912	25.10	1.50	14.45	8.4	24.35	0.75	0.74
1913	22.24	1.62	12.65	7.2	21.47	0.77	0.89
Total...	113.65	8.02	62.69	37.4	108.11	5.54	2.80
Mean...	22.7	1.6	12.5	7.5	21.6	1.1 § (0.77)	0.70

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

‡ The run-off at Mankato for this year was 3.4 in. On this basis, the run-off above Montevideo would probably be from 2.25 to 2.75 in.

§ Mean of years over which observations extend.

Note.—A run-off year beginning March 1st and ending on the last of February following, is assumed to correspond to the rainfall year beginning November 1st and ending on the following October 31st. It is assumed that practically all excess ground-water available for run-off on November 1st of any year will have appeared as run-off before the following March 1st. As no surface run-off appeared for November during the given years, the computed precipitation minus total loss for this water-shed is directly comparable with the actual run-off for the run-off year.

puted run-off data for any given water-shed are of service only as a basis for estimating the run-off which will probably occur in the future. An identical recurrence of any given combination of meteorological phenomena on any one water-shed is extremely improbable. In view of this fact, the complete daily, and to a large extent the monthly, distribution of run-off, is of much less importance than the annual yield of a water-shed, the probable extreme maximum flow, the extreme minimum, and a reasonably accurate estimate of run-off below the limit of economical utilization for whichever purpose the stream flow is to be used.

Inasmuch as the low-water flow from most small water-sheds is so extremely small as to be hardly capable of economical use except through storage reservoirs, the sudden fluctuations in stream flow below the maximum expected flood, with the reservoir filled, are of little consequence. Whether 1 in. of run-off occurs in a few days or in a few weeks is not of much consequence on such a water-shed, if all the available run-off can be held in the storage reservoir for gradual

TABLE 11.—DATA AND COMPUTATIONS FOR THE MINNESOTA RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Water Area.		Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Evaporation, in inches.		Transpiration equivalent, in inches.	Evaporation, in inches.			
			From curve.	Actual.		From curve.	Actual.		
1908.									
Nov.	35.5	1.89	.....	.....	.....	0.42	0.52	0.52	1.37
Dec.	18.0	1.38	.....	.....	.....	0.40	0.50	0.50	0.88
1909.									
Jan.	11.5	0.88	.....	.....	.....	0.25	0.31	0.31	0.57
Feb.	14.2	1.49	.....	.....	.....	0.30	0.37	0.37	1.12
Mar.	28.0	0.25	.....	.....	.....	0.45	0.56	0.56	−0.31
Apr.	37.5	0.87	2.20	0.11	.....	0.55	0.65	0.76	0.11
May.	55.0	4.81	4.20	0.21	1.20	2.00	2.37	3.78	1.03
June.	66.2	2.70	5.60	0.28	1.70	1.50	1.78	3.76	−1.06
July.	70.0	2.21	5.80	0.29	1.20	1.40	1.66	3.15	−0.94
Aug.	73.2	3.54	5.90	0.30	1.60	1.90	2.25	4.15	−0.61
Sept.	61.0	1.84	4.00	0.20	1.00	0.90	1.07	2.27	−0.43
Oct.	45.5	1.86	2.10	0.10	0.30	0.60	0.71	1.11	0.75
	.....	23.72	29.80	1.49	7.60	10.67	12.75	21.24	2.48
Nov.	33.0	1.32	1.10	0.06	.....	0.30	0.36	0.42	0.90
Dec.	10.0	1.75	.....	.....	.....	0.20	0.25	0.25	1.50
1910.									
Jan.	13.5	0.76	.....	.....	.....	0.30	0.37	0.37	0.39
Feb.	8.5	0.49	.....	.....	.....	0.25	0.31	0.31	0.18
Mar.	45.0	0.25	3.00	0.15	0.1	*0.15	0.18	0.43	−0.18
Apr.	49.0	2.28	3.50	0.17	0.2	*0.85	1.02	1.39	0.89
May.	53.5	0.92	4.00	0.20	0.7	*0.45	0.53	1.43	−0.51
June.	68.2	2.49	5.80	0.29	1.5	*1.35	1.60	3.39	−0.90
July.	72.3	1.49	6.30	0.32	1.0	*0.95	1.13	2.45	−0.96
Aug.	67.2	3.61	5.00	0.25	1.8	*1.65	1.96	4.01	−0.40
Sept.	60.0	1.01	3.80	0.19	0.8	*0.40	0.48	1.47	−0.46
Oct.	54.0	1.39	3.00	0.15	0.4	*0.55	0.65	1.20	0.19
	.....	17.76	35.50	1.78	6.5	7.40	8.84	17.12	0.64

* Low storage.

utilization. The engineer is usually much more interested in the total run-off from such a water-shed, up to the point of economical utilization, than in the exact distribution of that run-off through the year.

Although the writer does not pretend to be able to compute accurately the daily or even the monthly discharge of a stream, as previously stated, he does believe it to be possible to compute a reasonable

TABLE 12.—SUMMARY OF DATA AND COMPUTATIONS FOR THE ROOT RIVER WATER-SHED, ABOVE HOUSTON, MINN.

Area = 1 560 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Computed run-off.†	Observed run-off.‡
1908	34.8	19.0	8.9	27.9	6.9	....	....
1909	33.2	17.3	9.0	26.3	6.9	4.32§	4.10§
1910	23.2	11.2	7.7	18.9	4.3	4.45	4.22
1911	35.0	18.4	9.2	27.6	7.4	5.28	5.19
1912	29.9	15.6	9.5	25.1	4.8	5.82	5.68
1913	31.9	17.3	9.1	26.4	5.5	5.08	5.73
Total.	188.0	98.8	53.4	152.2	35.8	20.63	20.82
Mean.	31.4	16.5	8.9	25.4	6.0	5.16	5.20

* From November 1st of previous year to October 31st of given year.

† Sum of monthly run-off from March 1st of given year to February 28th or 29th of following year, as computed in Table 35.

‡ From March 1st of given year to February 28th or 29th of following year.

§ From June to February, inclusive.

NOTE.—This water-shed is in the southeastern corner of Minnesota, and the winter temperatures are not sufficiently low to prevent surface run-off, consequently the annual precipitation minus losses figured for the rainfall year beginning Nov. 1st does not check so well with the observed run-off for the year beginning March 1st, as it does on more northerly water-sheds. It will be noted, however, that the computed run-off checks the observed run-off very closely.

distribution of the annual yield of a given water-shed through the various months of the year by making a proper study of the factors that influence this distribution of run-off. To do this requires considerable expenditure of time and the use of data seldom readily available.

In the first place, it is necessary to have detailed knowledge of the physical characteristics of the water-shed under consideration, including topography, geology, vegetation, soil, etc., and the character and daily rate of precipitation. It is important to know whether a given winter's snowfall will run off suddenly during a heavy spring rain, or whether it will melt gradually and be largely absorbed by the soil. It is important to know whether, after heavy rains, a large portion of the precipitation immediately appears in the streams or whether it is paid out gradually from lakes and swamps. It is important to know whether, during the summer, when the demands of evaporation and transpiration absorb most of the available rainfall.

TABLE 13.—Summary of Data and Computations for the Ottertail River Water-shed, Above Fergus Falls, Minn.

Area = 1 310 sq. miles.

Year.	Rainfall.*	Evaporation.		Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.†
		Water.	Land.						
1908.....	25.1	3.4	10.9	6.8	21.1	4.0	0	4.0	3.84‡
1909.....	25.4	3.6	10.9	6.9	21.4	4.0	0.2	3.8	3.90§
1910.....	15.5	3.7	5.9	4.9	14.5	1.0	—0.8	1.8	1.79
1911.....	22.8	3.5	10.1	7.0	20.6	2.2	0.6	1.6	1.43
1912.....	23.6	3.2	11.0	7.2	21.4	2.2	0.1	2.1	1.97
1913.....	25.8	3.4	11.3	7.0	21.7	4.1	0.3	3.8	
Total.....	138.2	20.8	60.1	39.8	120.7	17.5	.....	17.1	12.93
Mean.....	23.0	3.5	10.0	6.6	20.1	2.9	.....	2.8 (2.66)‖	2.59

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

‡ January and February estimated at 0.34.

§ March estimated at 0.25.

‖ Mean of years over which observations extend.

Note.—In view of the large number of lakes on this water-shed, some change in storage is certain to occur, notwithstanding the equalization of ground-water which may be effected during the period between the close of the rainfall year and the beginning of the run-off year.

the stream will be fed by an abundant ground-water storage. It is important to know whether the temperature during the winter suddenly drops to a point where a heavy ice cover is formed over the stream in one or two days, almost shutting off the flow until the stage has sufficiently increased and the slope has been sufficiently equalized and increased, so that the combined area of cross-section and gradient are ample to overcome the increased friction due to ice cover, and thus carry approximately the quantity of water that was flowing in the stream before the freeze-up. It is important to know, further, whether the ground-water table lies so close to the surface that a large portion of the ground-water which would otherwise maintain stream flow during the winter, is frozen up and held until the following spring. It is important to know the absorptive capacity of the soil on a given water-shed, and the character of the underlying soil or rock. It is important to know whether, at a given point, a stream flows over a bed of impervious rock which underlies

TABLE 14.—DATA AND COMPUTATIONS FOR THE OTTERTAIL WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches	Loss from Water Area.		Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Evaporation, in inches.		Transpiration equivalent, in inches.	Evaporation, in inches.			
			From curve.	Actual.		From curve.	Actual.		
1907.									
Nov.	29	0.2	1.00	0.15	.....	0.04	0.04	0.19	0.01
Dec.	19	0.5	.....	.....	.....	0.22	0.24	0.24	0.26
1908.									
Jan	14	0.2	.....	.....	.....	*0.18	0.20	0.20	0.00
Feb	15	1.6	.....	.....	.....	0.40	0.44	0.44	1.16
Mar	22	1.5	.....	.....	.....	0.60	0.66	0.66	0.84
Apr	44	1.4	1.70	0.26	0.1	0.70	0.65	1.01	0.39
May	53	5.2	2.46	0.37	1.0	2.08	1.95	3.32	1.88
June	62	5.5	3.33	0.50	1.5	2.50	2.34	4.34	1.16
July	70	2.6	4.30	0.65	1.5	1.60	1.50	3.65	— 1.05
Aug.	65	3.0	3.90	0.58	1.5	1.50	1.40	3.48	— 0.48
Sept.	63	2.2	3.70	0.55	1.0	1.15	1.08	2.63	— 0.43
Oct	45	1.2	2.10	0.32	0.2	0.41	0.38	0.90	0.30
	.....	25.1	22.49	3.38	6.8	11.38	10.88	21.06	4.04
Nov.	32	1.7	1.15	0.17	.....	0.35	0.33	0.50	1.20
Dec.	13	0.7	.....	.....	.....	0.27	0.30	0.30	0.40
1909.									
Jan	7	0.9	.....	.....	.....	0.14	0.16	0.16	0.74
Feb	10	0.6	.....	.....	.....	0.21	0.23	0.23	0.37
Mar	24	0.3	0.88	0.13	.....	0.50	0.47	0.60	— 0.30
Apr	34	1.5	1.80	0.27	.....	*0.60	0.56	0.83	0.67
May	53	3.9	2.46	0.37	1.0	1.66	1.55	2.92	0.98
June	65	2.7	3.65	0.55	1.5	1.58	1.48	3.53	— 0.83
July	67	4.5	4.00	0.60	1.6	2.30	2.15	4.35	0.15
Aug.	71	5.1	4.50	0.68	1.6	2.51	2.35	4.63	0.47
Sept.	59	1.8	3.33	0.50	1.0	0.89	0.83	2.33	— 0.53
Oct	43	1.7	1.95	0.29	0.2	0.54	0.50	0.99	0.71
	.....	25.4	23.72	3.56	6.9	11.55	10.91	21.37	4.03

*Low storage.

the entire water-shed, thus preventing all possible deep seepage losses, or whether it flows over a bed of sand and gravel overlying pervious sandstone, thus admitting of considerable underflow and deep seepage loss. It is important to know whether the stream flow consists largely of seepage flow, or whether practically all the discharge represents surface run-off resulting from excessive precipitation over restricted areas. These are but a few of the most important considerations which must be kept in mind in estimating the distri-

TABLE 15.—Summary of Data and Computations for the St. Croix River Water-shed, Above St. Croix Falls, Wis.

Area = 5 930 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Surface run-off.	Change in storage.	Computed run-off.	Observed run-off.†
1902	24.66	12.0	6.8	18.8	5.9	0.4	−0.2	6.5	6.65
1903	44.16	16.7	7.7	24.4	19.7	−0.2	2.0‡	17.5	16.14
1904	32.64	13.1	7.1	20.2	12.4	0.2	−0.4	13.0	11.85
1905	32.67	14.5	6.9	21.4	11.3	−0.2	−0.5	11.6	11.73
1906	33.17	14.5	7.0	21.5	11.7	0.0	−0.2	11.9	10.95
1907	29.18	12.1	6.4	18.5	10.7	−0.2	0.6	9.9	10.45
1908	30.57	13.8	6.9	20.7	9.9	0.0	−0.2	10.1	10.02
1909	27.83	12.9	6.7	19.6	8.2	0.4	−0.4	9.0	9.44
1910	18.64	8.0	6.1	14.1	4.5	−0.4	−1.5	5.6	4.86
1911	30.68	14.6	8.1	22.7	8.0	0.0	1.0	7.0	5.61
1912	26.48	12.3	7.2	19.5	7.0	0.0	−0.4	7.4	6.92
Total...	330.68	144.5	76.9	221.4	109.3	.....	.....	109.5	105.62
Mean..	30.0	13.1	7.0	20.1	9.9	.....	.....	9.9	9.6

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

‡ 24 in. of rainfall from July to October, inclusive.

Note.—Large portions of the St. Croix River water-shed are quite sandy, and the topography is such that considerable quantities of water are stored for longer periods than that represented between the close of the rainfall year, November 1st, and the beginning of the next run-off year, March 1st; consequently, for this water-shed, some allowance has been made for a change in storage. Except in one or two years, however, the estimated change in storage has not affected the computed run-off materially. That considerable changes in storage occurred during the years for which such allowance has been made, is substantiated by the variation in actual February run-off.

bution of run-off, as well as in determining the annual yield of a water-shed.

To aid in computing the monthly distribution of annual run-off, diagrams of the type shown in Figs. 30 to 32 for the Root River water-shed have been prepared. These are to be used in conjunction with daily temperature and precipitation records. The curves of Fig. 30 show, in the first place, the approximate maximum quantity of snow which, when available, will melt at the given monthly mean temperatures. The other curves in Fig. 30 give the estimated quantity of this melted snow which will percolate into the ground for various soil storage capacities. By "soil storage capacity" is to be understood, in this case, the approximate quantity of water, in inches of depth, which the upper foot or so of soil is capable of absorbing at

TABLE 16.—DATA AND COMPUTATIONS FOR THE ST. CROIX RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1906.							
Nov.	31.4	2.38	.....	0.50	0.52	0.52	1.86
Dec.	17.2	1.34	.....	0.40	0.42	0.42	0.92
1907.							
Jan.	5.9	1.47	.....	0.10	0.10	0.10	1.37
Feb.	14.0	0.72	.....	0.35	0.37	0.37	0.35
Mar.	27.4	1.73	.....	0.70	0.74	0.74	0.99
Apr.	34.9	0.60	.....	0.50	0.52	0.52	0.08
May	43.7	2.99	0.4	1.15	1.21	1.61	1.38
June	61.8	3.54	1.5	1.75	1.84	3.34	0.20
July	66.4	3.17	1.7	1.78	1.87	3.57	—0.40
Aug.	64.8	2.73	1.5	1.40	1.47	2.97	—0.24
Sept.	55.3	7.77	1.2	2.60	2.73	3.93	3.84
Oct.	43.8	0.74	0.1	0.30	0.32	0.42	0.32
	.....	29.18	6.4	11.53	12.11	18.51	10.67
Nov.	21.7	1.75	.....	0.43	0.45	0.45	1.30
Dec.	21.5	1.53	.....	0.45	0.47	0.47	1.06
1912.							
Jan.	— 6.7	0.50	.....	0.05	0.05	0.05	0.45
Feb	10.6	0.23	.....	0.27	0.23	0.26	—0.03
Mar.	20.8	0.48	.....	0.47	0.49	0.49	—0.01
Apr	46.6	2.43	0.2	1.05	1.10	1.30	1.13
May	56.4	6.58	1.1	2.57	2.70	3.80	2.78
June	61.8	1.84	1.5	1.10	1.16	2.66	—0.82
July	68.9	3.47	1.7	1.95	2.05	3.75	—0.28
Aug.	63.8	4.89	1.7	2.12	2.23	3.93	0.96
Sept.	59.0	1.85	0.9	0.90	0.95	1.85	0.00
Oct.	48.1	0.93	0.1	0.38	0.40	0.50	0.43
	.....	26.48	7.2	11.72	12.31	19.51	6.97

any given time. The soil storage capacity on any given water-shed at the time of break-up in the spring, for example, will depend not only on the character of the soil, but on the quantity of water held over from the preceding fall, in the upper layers of soil, because the ground remained frozen through the winter. A portion of the snow which does not percolate into the ground, on melting, will immediately run off into the streams. Another portion will be retained for some time, part to appear as run-off later, and part to be gradually absorbed by the soil. The proportion retained for a time depends largely on topographic conditions. It may be treated substantially as precipi-

tation minus losses for the following month, resulting from well-distributed rains.

The curves of Fig. 31 are to be used to aid in determining the quantity of surface run-off resulting from a given monthly "precipitation minus losses". These curves must also be used in connection with daily precipitation records. In Minnesota, less surface run-off will, in general, result from a given "precipitation minus losses" in spring and fall than in summer, because the rains are usually well distributed during the former seasons. Some latitude must be allowed in the application of these curves.

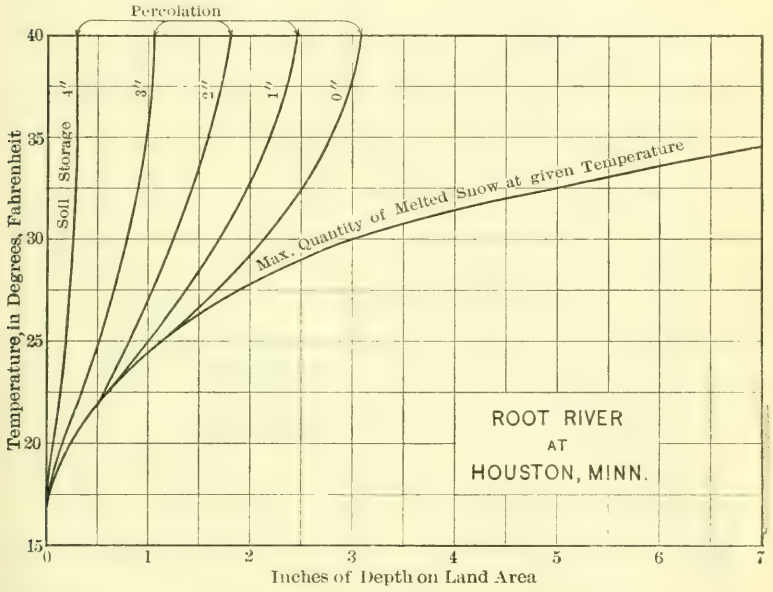


FIG. 30.

The curve of Fig. 32 is to be used to determine the seepage flow for a given quantity of subsoil storage. On the Root River water-shed, moisture which has once passed down through the upper foot or two of soil will continue downward, as a rule, to join the subsoil storage and aid in maintaining stream flow. It is practically safe against return through the action of capillarity.

The shape of the curves for other water-sheds of the type shown for the Root River water-shed varies mostly with topography, character of subsoil, and size of drainage basin.

In order to show the application of the writer's method of computing the monthly distribution of annual run-off, the detailed computations for the Root River water-shed at Houston, Minn., are given in Table 35. Summing up the monthly run-off figures as computed in this way, gives a value for the annual run-off which is much more accurate, and differs somewhat, it will be noted, from the computed

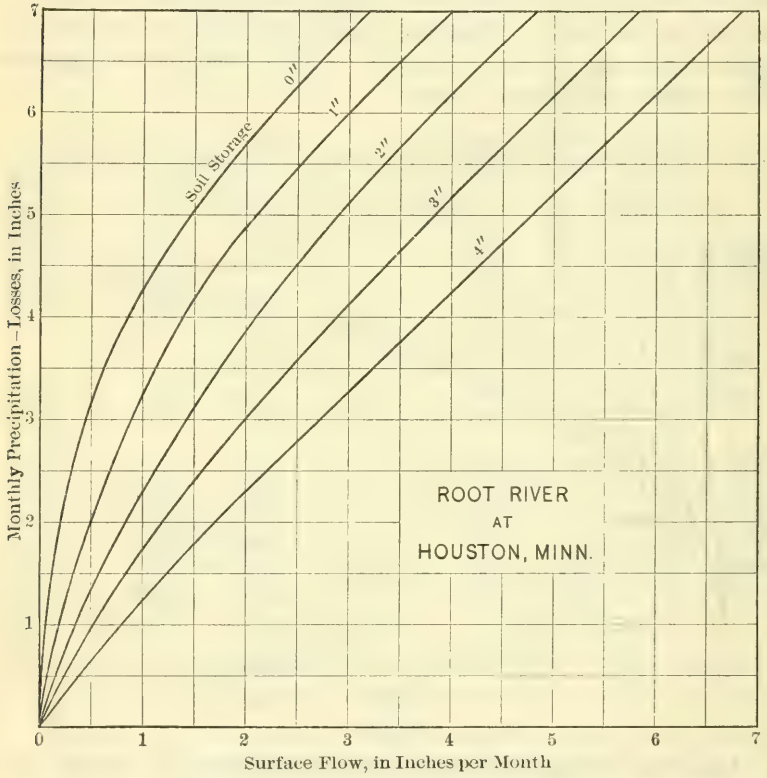


FIG. 31.

annual "precipitation minus losses" given in the summary of data and computations for the Root River water-shed. The difference results from surface run-off between November 1st and March 1st, and from differences in ground-water storage at the close of the water year. The average annual computed "precipitation minus losses" for the 4 years over which discharge records extend agrees substantially with the average annual observed run-off.

Frequency Curves.

The curves of Fig. 33 are presented in order to show the great difference in the annual run-off from the Mississippi River and the Tohickon Creek water-sheds, and its distribution.

The most striking facts, perhaps, are that the maximum rate of run-off, in inches of depth per month, for the Tohickon Creek water-shed is about four and one-half times that for the Mississippi River

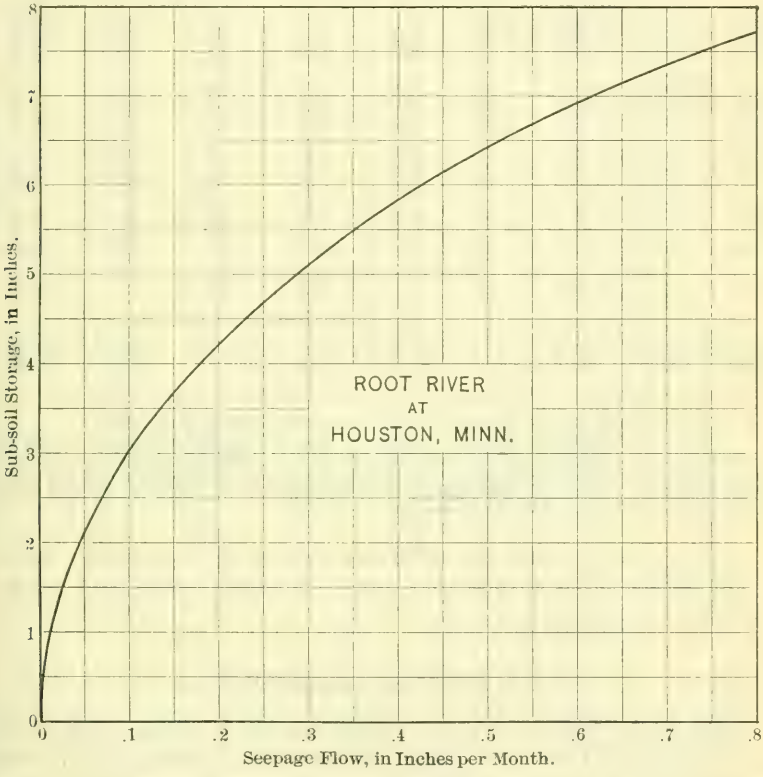


Fig. 32.

water-shed, and that the minimum rate of run-off from the former water-shed is about one-half the minimum rate from the latter. For about 10% of the time the rate of run-off from the Mississippi River water-shed is greater than that of the Tohickon Creek, in spite of the fact that the annual yield of the former water-shed is only about one-fifth of that of the latter.

TABLE 17.—SUMMARY OF DATA AND COMPUTATIONS FOR THE OHIO RIVER WATER-SHED, ABOVE WHEELING, W. VA.

Area = 23 820 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1892....	42.53	15.5	5.9	21.4	21.1	—0.3	21.4	22.01
1893....	41.95	14.75	5.8	20.55	21.4	—0.2	21.6	21.93
1894....	37.17	13.9	5.4	19.3	17.9	0.5	17.4	17.78
1895....	32.07	11.8	5.8	17.6	14.5	—0.4	14.9	16.14
1896....	45.79	16.8	6.1	22.9	22.9	0.5	22.4	21.24
1897....	36.60	13.5	5.7	19.2	17.4	—0.7	18.1	21.03
1898....	50.66	17.5	6.1	23.6	27.0	1.1	25.9	25.64
1899....	40.30	14.5	6.0	20.5	19.8	—0.8	20.6	21.93
1900....	37.02	13.7	5.9	19.6	17.4	0.3	17.1	17.60
1901....	43.15	15.8	5.7	21.5	21.7	—0.6	22.3	24.72
1902....	43.22	15.1	5.8	20.9	22.3	0.8	21.5	25.08
1903....	45.08	15.5	5.8	21.3	23.8	—0.1	23.9	27.08
1904....	40.73	13.9	5.9	19.8	20.9	—0.2	21.1	24.99
1905....	40.00	15.0	6.0	21.0	19.0	0.0	19.0	21.20
Total...	576.27	207.25	81.9	289.1	287.1	.....	287.1	308.37
Mean ..	41.1	14.8	5.8	20.6	20.5	.....	20.5	22.0

* From November 1st of previous year to October 31st of given year.

NOTE.—It is believed that the location of Weather Bureau stations in cities and villages, as a rule situated on railroads, results in temperature records for this water-shed which are somewhat too large, and precipitation records which are somewhat too small. Any such discrepancy in fundamental data would be reflected in the computed run-off, as in fact it appears to be.

These curves reflect the difference in size of water-sheds, besides many of the other factors that affect the annual yield, and its distribution throughout the year.

SEEPAGE FLOW AND SURFACE RUN-OFF.

The writer believes that the percentage which seepage flow constitutes of the total run-off on most streams of the Northwest is not fully realized. It is safe to say that, but for a few exceptional years, practically all the water in Minnesota streams from the first of November to the first of March is seepage flow. During the break-up and the spring rains following it, the ground-water storage which sustained the stream flow during the winter is again replenished. During July and August the demands of evaporation and transpiration consume practically all the available rainfall, so that the stream flow is again main-

TABLE 18.—Data and Computations for the Ohio River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1894.							
Nov.	40	2.54	.....	0.70	0.61	0.61	1.93
Dec.	36	3.82	.....	0.85	0.74	0.74	3.08
1895.							
Jan.	27	4.22	.....	0.95	0.83	0.83	3.39
Feb.	24	1.19	.....	0.60	0.52	0.52	0.67
Mar.	40	2.34	.....	0.90	0.79	0.79	1.55
Apr.	53	2.68	0.2	1.20	1.05	1.25	1.43
May	62	2.18	0.8	1.20	1.05	1.85	0.33
June.	73	3.12	1.3	1.90	1.66	2.96	0.16
July	70	3.38	1.4	1.80	1.57	2.97	0.41
Aug	72	3.41	1.3	1.85	1.62	2.92	0.49
Sept.	69	2.06	0.7	1.20	1.05	1.75	0.31
Oct.	48	1.13	0.1	0.40	0.35	0.45	0.68
	.....	32.07	5.8	13.55	11.84	17.64	14.43
Nov.	40	3.55	.....	0.90	0.79	0.79	2.76
Dec.	38	3.57	.....	0.85	0.74	0.74	2.83
1896.							
Jan.	32	1.72	.....	0.75	0.66	0.66	1.06
Feb.	33	3.21	.....	1.00	0.88	0.88	2.33
Mar.	35	3.73	.....	1.10	0.96	0.96	2.77
Apr.	58	2.70	0.2	1.40	1.23	1.43	1.27
May	69	2.97	0.8	1.75	1.53	2.33	0.64
June.	70	5.23	1.4	2.70	2.36	3.76	1.47
July	73	9.08	1.6	4.25	3.72	5.32	3.76
Aug	72	2.83	1.3	1.60	1.40	2.70	0.13
Sept.	64	4.85	0.7	2.10	1.84	2.54	2.31
Oct.	49	2.35	0.1	0.80	0.70	0.80	1.55
	.....	45.79	6.1	19.20	16.81	22.91	22.88

tained by the ground-water supply. In September and October, fall rains, though largely absorbed, nevertheless usually supply a certain quantity of surface run-off. It is believed that on an average one-half to three-fourths of the annual run-off may be considered as being seepage flow.

On water-sheds such as that of the Colorado River in Texas, where the precipitation is distributed throughout the year in a similar manner to that prevailing throughout the Northwest, but where temperatures are so high that evaporation losses absorb by far the greater portion of the rainfall, and where the demands of vegetation for

moisture are never fully supplied, most of the stream flow results from surface run-off. On such water-sheds as that of the Colorado, practically all stream flow above a few hundredths of an inch of depth per month results from heavy rains commonly called "cloudbursts" over

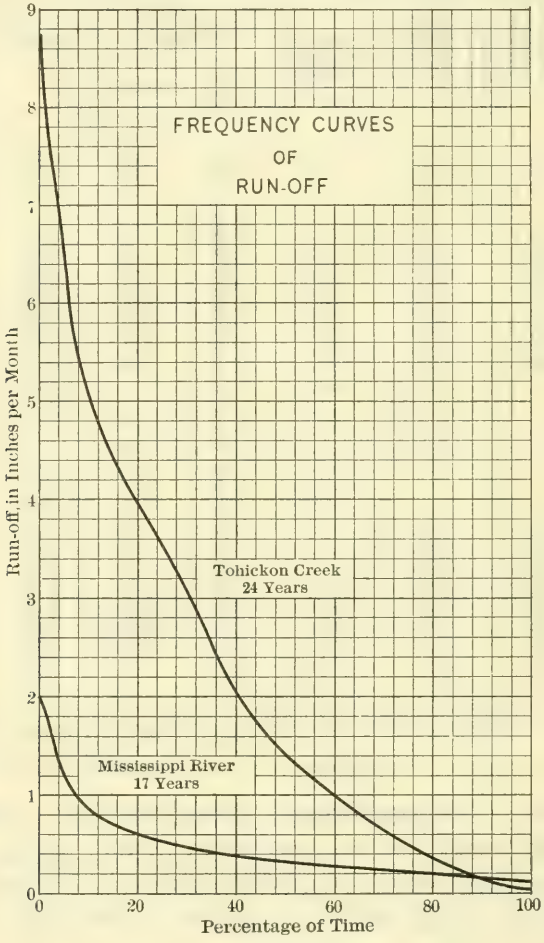


FIG. 33.

small areas. As the surface of the ground on such a water-shed is usually very dry, percolation is not very rapid, hence some of the rainfall escapes over the surface of the ground as run-off before sufficient time has elapsed for it to be evaporated, or to be used by vegetation.

TABLE 19.—Summary of Data and Computations for the Tohickon Creek Water-shed, Above Point Pleasant, Pa.

Area = 102 sq. miles.

Year.	Rainfall.*	Evaporation	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1888	47.9	15.0	6.5	21.5	26.4	30.4
1889	65.8	21.8	7.5	29.3	36.5	40.3
1890	63.1	20.0	7.6	27.6	35.5	35.8
1891	55.0	18.2	7.1	25.3	29.7	30.6
1892	41.8	14.6	6.7	21.3	20.5	23.4
1893	48.3	15.6	6.6	22.2	26.1	27.7
1894	45.2	15.1	6.5	21.6	23.6	26.6
1895	51.4	17.9	6.7	24.6	26.8	29.5
1896	42.2	14.0	7.5	21.5	20.7	15.6
1897	51.7	19.3	7.7	27.0	24.7	23.5
1898	46.0	16.4	6.7	23.1	22.9	23.8
1899	48.3	15.7	6.6	22.3	26.0	31.8
1900	44.2	16.0	7.5	23.5	20.7	19.2
1901	40.5	15.5	7.1	22.6	17.9	17.1
1902	44.7	14.2	6.9	21.1	23.6	27.3
1903	61.2	19.7	7.0	26.7	34.5	37.6
1904	48.8	17.1	7.0	24.1	24.7	25.5
1905	44.6	15.8	6.8	22.6	22.0	22.8
1906	48.7	17.6	7.6	25.2	23.5	23.2
1907	43.1	15.1	6.5	21.6	21.5	22.0
1908	62.4	20.6	7.5	28.1	34.3	33.3
1909	39.6	14.6	6.8	21.4	18.2	17.0
1910	42.4	14.5	5.9	20.4	22.0	24.5
1911	46.5	17.7	7.3	25.0	21.5	17.4
Total..	1 173.4	402.0	167.6	569.6	603.8	625.9
Mean..	48.9	16.7	7.0	23.7	25.2	26.1

* From September 1st of previous year to August 31st of given year.

Note.—Excessive rains occurred during August, 1911; consequently, the computed precipitation minus losses for the water year contains at least about 2 in. of water which did not appear as run-off during that month, so that the computed run-off would probably be about 19.5 in. In no other year did a similar condition occur, so that the computed precipitation minus losses is believed to represent the run-off during the same water year within a few tenths of an inch.

Flood Flows.

It is impossible to foretell the exact combination of physical conditions which will result in the maximum flood in any given drainage basin. The maximum spring flood on such a water-shed as that of the Root River would occur at approximately such a time as the middle of March, 1913, when the temperature suddenly rose from relatively cold to considerably above freezing all day, and when several inches of warm rains, precipitated on frozen ground covered with the winter's

accumulation of snow, resulted in sending most of the surface storage plus the precipitation into the stream in a few days.

The conditions resulting in maximum floods on different water-sheds vary considerably, and each stream must be studied in detail as a separate problem. Unless the observed stream-flow data extend over a long period of years, the existence of discharge records for the given stream will not obviate the necessity for such study. On the larger Minnesota water-sheds, such as that of the Mississippi, local differences in temperature make early spring floods much less frequent than on smaller water-sheds, such as that of the Root River. The highest flood on the Mississippi at Minneapolis will probably occur in May or June, after general heavy precipitation throughout the water-shed, following a fall, winter, and spring which favored percolation, and thus exhausted all the available ground and lake storage.

TABLE 20.—Data and Computations for the Tohickon Creek Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.	Recorded run-off.
			Transpiration, in inches.	Evaporation, in inches.				
				From curve.	Actual.			
1898.								
July	76.9	4.0	1.5	2.42	2.18	3.68	0.32	0.08
Aug	74.1	6.1	1.5	3.05	2.75	4.25	1.85	0.75
Sept	68.3	2.0	0.9	1.15	1.03	1.93	0.07	0.09
Oct	56.0	5.2	0.4	1.95	1.76	2.16	3.04	0.61
Nov	42.6	7.0	.....	1.75	1.58	1.58	5.42	4.49
Dec	32.5	3.5	.....	0.70	0.63	0.63	2.87	4.24
1899.								
Jan	28.2	3.7	.....	0.92	0.83	0.83	2.87	4.75
Feb	25.2	4.8	.....	0.90	0.81	0.81	3.99	5.63
Mar	38.2	6.6	.....	1.70	1.53	1.53	5.07	9.01
Apr	50.1	2.2	0.3	1.00	0.90	1.20	1.00	1.58
May	60.6	2.2	0.9	1.20	1.08	1.98	0.22	0.25
June	71.1	2.7	1.3	1.65	1.48	2.78	—0.08	0.08
July	73.6	3.3	1.4	2.00	1.80	3.20	0.10	0.08

Note.—In order to get a better comparison between the computed and recorded run-off for the year 1898 to 1899, the computations are given for the 13 months beginning July, 1898, and ending July, 1899. It will be noted that the recorded run-off for July of both years is exactly the same, and very small, indicating equalization of ground storage between these dates, and thus affording a better comparison between the computed and recorded run-off. It is worthy of note that the rainfall from October to March, inclusive, was 30.8 in., and the recorded run-off was 28.73 in., or 93 per cent. Assuming the records to be correct, this would indicate an evaporation loss of less than 0.4 in. per month for temperatures ranging from 25 to 56 degrees. As the evaporation was unquestionably at least twice as much, there can be no doubt that either the rainfall or run-off records are decidedly in error.

TABLE 20.—(*Continued.*)

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1900.							
Sept	69	1.6	0.9	0.95	0.86	1.76	−0.16
Oct	59	3.0	0.3	1.32	1.19	1.49	1.51
Nov	46	2.6	.....	0.85	0.77	0.77	1.83
Dec	32	2.6	.....	0.52	0.47	0.47	2.13
1901.							
Jan	30	2.6	.....	0.85	0.77	0.77	1.83
Feb	23	0.9	.....	0.52	0.47	0.47	0.43
Mar	39	4.8	.....	1.40	1.26	1.26	3.54
Apr	49	5.7	0.3	2.05	1.85	2.15	3.55
May	59	5.2	1.1	2.30	2.07	3.17	2.03
June	69	1.8	1.4	1.20	1.08	2.48	−0.68
July	78	2.6	1.6	1.75	1.58	3.18	−0.58
Aug	74	7.1	1.5	3.45	3.10	4.60	2.50
	.....	40.5	7.1	17.16	15.47	22.57	17.93
Sept	66	2.5	1.0	1.30	1.07	2.07	0.43
Oct	54	1.5	0.3	0.68	0.61	0.91	0.59
Nov	38	2.0	.....	0.52	0.47	0.47	1.53
Dec	31	7.8	.....	1.05	0.94	0.94	6.86
1902.							
Jan	28	3.1	.....	0.85	0.76	0.76	2.34
Feb	26	5.6	.....	0.95	0.85	0.85	4.75
Mar	43	4.2	.....	1.40	1.26	1.26	2.94
Apr	50	3.9	0.3	1.56	1.40	1.70	2.20
May	60	1.6	0.9	0.95	0.85	1.75	−0.15
June	68	5.1	1.5	2.58	2.32	3.82	1.28
July	73	4.2	1.6	2.40	2.16	3.76	0.44
Aug	70	3.2	1.3	1.70	1.53	2.83	0.37
	.....	44.7	6.9	15.94	14.22	21.12	23.58
Sept	64	7.6	1.0	3.05	2.75	3.75	3.85
Oct	55	5.8	0.3	2.05	1.85	2.15	3.65
Nov	49	1.7	.....	0.62	0.56	0.56	1.14
Dec	29	7.6	.....	0.92	0.83	0.83	6.77
1903.							
Jan	29	4.3	.....	1.05	0.94	0.94	3.36
Feb	32	5.1	.....	1.22	1.10	1.10	4.00
Mar	48	4.2	.....	1.60	1.44	1.44	2.76
Apr	51	4.6	0.3	1.86	1.62	1.92	2.68
May	64	0.5	0.7	0.45	0.40	1.10	−0.60
June	65	8.6	1.7	3.60	3.24	4.94	3.66
July	73	6.4	1.7	3.25	2.93	4.63	1.77
Aug	69	4.8	1.3	2.30	2.07	3.37	1.43
	.....	61.2	7.0	21.91	19.73	26.73	34.47

TABLE 20.—(*Continued.*)

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1906.							
Sept.	68	1.7	0.8	1.00	0.90	1.70	0.00
Oct.	55	6.7	0.4	2.30	2.07	2.47	4.23
Nov.	43	1.6	.....	0.50	0.45	0.45	1.15
Dec.	31	4.6	.....	0.85	0.77	0.77	3.83
1907.							
Jan.	30	4.0	.....	1.00	0.90	0.90	3.10
Feb.	28	2.8	.....	0.70	0.63	0.63	2.17
Mar.	43	3.2	.....	1.20	1.08	1.08	2.12
Apr.	47	3.4	0.2	1.30	1.17	1.37	2.03
May.	57	3.5	0.8	1.65	1.48	2.28	1.22
June.	65	5.1	1.5	2.50	2.25	3.75	1.35
July.	74	3.0	1.5	1.90	1.71	3.21	—0.21
Aug.	70	3.5	1.3	1.85	1.66	2.96	0.54
	.....	43.1	6.5	16.75	15.07	21.58	21.53

TABLE 21.—Summary of Data and Computations for the James River Water-shed, Above Cartersvile, Va.

Area = 6 230 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1899	44.5	16.4	7.1	23.5	21.00	—1.4	22.4	20.3
1900	37.7	15.3	6.5	21.8	15.90	0.6	15.3	14.0
1901	51.6	20.5	7.3	27.8	23.80	—1.0	24.8	24.0
1902	43.3	14.2	7.0	21.2	22.10	1.5	20.6	19.6
1903	46.9	17.5	7.1	24.6	22.30	—1.0	23.3	23.9
1904	28.7	12.6	6.7	19.3	9.40	—0.5	9.9	10.3
1905	41.7	17.3	7.6	24.9	16.80	0.6	16.2	14.1
Total.	294.4	113.8	49.3	163.1	131.30	.....	132.5	126.2
Mean.	42.1	16.3	7.0	23.3	18.8	.....	18.9	18.0

* From November 1st of previous year to October 31st of given year.

TABLE 22.—DATA AND COMPUTATIONS FOR THE JAMES RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1899.							
Nov	45.8	0.93	.....	0.35	0.32	0.32	0.61
Dec	34.9	1.90	.....	0.45	0.42	0.42	1.48
1900.							
Jan.....	34.6	3.35	.....	1.01	0.94	0.94	2.41
Feb.....	32.7	4.41	.....	1.16	1.07	1.07	3.34
Mar	40.8	3.76	.....	1.25	1.16	1.16	2.60
Apr	54.1	2.84	0.4	1.31	1.21	1.61	1.23
May....	63.4	2.92	1.1	1.60	1.48	2.58	0.34
June...	69.9	5.34	1.5	2.72	2.52	4.02	1.32
July....	75.6	2.98	1.4	1.88	1.74	3.14	—0.16
Aug....	78.2	1.54	1.0	1.10	1.02	2.02	—0.48
Sept ...	71.2	3.98	0.8	2.05	1.90	2.70	1.28
Oct.....	60.3	3.75	0.3	1.62	1.50	1.80	1.95
	.....	37.70	6.5	16.50	15.28	21.78	15.92
Nov.....	47.5	3.12	.....	1.05	0.97	0.97	2.15
Dec.....	36.7	2.78	.....	0.69	0.64	0.64	2.14
1901.							
Jan.....	35.3	2.67	.....	0.93	0.86	0.86	1.81
Feb	31.4	0.59	.....	0.50	0.46	0.46	0.13
Mar	44.6	3.48	.....	1.30	1.20	1.20	2.28
Apr	48.9	6.92	0.3	2.35	2.17	2.47	4.45
May....	61.9	6.70	1.1	2.90	2.68	3.78	2.92
June...	70.5	5.62	1.6	2.87	2.66	4.26	1.36
July....	77.0	5.23	1.7	2.99	2.77	4.47	0.76
Aug....	72.2	10.22	1.5	4.45	4.12	5.62	4.60
Sept....	64.8	3.75	0.8	1.80	1.67	2.47	1.28
Oct.....	54.0	0.55	0.3	0.30	0.28	0.58	—0.03
	.....	51.63	7.3	22.13	20.48	27.78	23.85

The determination of the probable extremes of discharge for streams on which only short-term run-off records are available is too large a problem to be discussed in the present paper.

NEED OF SUPPLEMENTING OBSERVED STREAM-FLOW DATA.

On comparatively few streams of the country do the records of discharge extend over a long term of years. Short-term records do not give the extremes of high and low flow unless by sheer accident such

TABLE 23.—SUMMARY OF DATA AND COMPUTATIONS FOR THE ROANOKE RIVER WATER-SHED.

Area = 390 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1897.....	38.1	15.4	6.8	22.2	15.9	0.8	15.1	15.22
1898.....	40.7	17.2	7.0	24.2	16.5	1.2	15.3	14.44
1899.....	44.1	16.7	7.0	23.7	20.4	—1.8	22.2	24.52
1900.....	40.8	16.8	6.9	23.7	17.1	0.6	16.5	13.19
1901.....	54.9	21.6	7.6	29.2	25.7	—0.8	26.5	29.32
1902.....	38.0	13.8	6.2	20.0	18.0	0.5	17.5	18.17
1903.....	47.5	18.1	7.0	25.1	22.4	—0.5	22.9	21.36
1904.....	33.2	14.0	7.0	21.0	12.2	—0.7	12.9	8.54
1905.....	45.8	18.3	7.5	25.8	20.0	1.0	19.0	14.65
Total....	383.1	151.9	63.0	214.9	168.2	.....	167.9	159.41
Mean....	42.6	16.9	7.0	23.9	18.7	.....	18.6	17.7

* From November 1st of previous year to October 31st of given year.

years have been included in the term over which observations extend. Short-term records, moreover, do not give a satisfactory value for mean utilizable flow. In the last analysis, it is usually necessary to supplement the observed stream-flow data with computed values based on rainfall and other physical data, in order to arrive at a probable maximum, minimum, and mean utilizable flow for any given stream.

In order to show the annual and periodic variations in rainfall and run-off on the two streams, considered in this paper, for which relatively long-term records are available, the curves in Figs. 34 to 36 have been prepared.

The curves of cumulative mean rainfall and run-off represent, at every point, the mean of all the annual values preceding. It is interesting to note that on the Mississippi River water-shed the 12-year mean run-off differs from the 6-year mean by more than 20 per cent. The 12-year mean rainfall, however, differs from the 6-year mean by only a small percentage. On the Tohickon Creek water-shed the 5-year mean rainfall is 111% of the 24-year mean, and the 5-year mean run-off is 123% of the long-term mean. Though the variation in run-off is proportionately very much larger than the variation in

TABLE 24.—Data and Computations for the Roanoke River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1896.							
Nov.....	52.2	4.47	.....	1.55	1.39	1.39	3.08
Dec.....	39.0	0.50	.....	0.18	0.16	0.16	0.34
1897.							
Jan.....	33.6	1.64	.....	0.40	0.36	0.36	1.28
Feb.....	40.6	7.10	.....	1.99	1.79	1.79	5.31
Mar	50.6	3.74	.....	1.55	1.39	1.39	2.35
Apr.....	55.6	1.67	0.4	0.92	0.83	1.23	0.44
May	63.0	3.71	1.0	1.90	1.71	2.71	1.00
June....	73.1	1.90	1.4	1.33	1.20	2.60	—0.70
July....	76.6	4.72	1.6	2.74	2.46	4.06	0.66
Aug.....	75.2	2.83	1.3	1.75	1.58	2.88	—0.05
Sept....	70.6	1.96	0.8	1.20	1.08	1.88	0.08
Oct.....	60.2	3.86	0.3	1.65	1.48	1.78	2.08
	.....	38.10	6.8	17.16	15.43	22.23	15.87
Nov	48.6	1.13	.....	0.46	0.42	0.42	0.71
Dec.....	40.6	3.72	.....	1.00	0.90	0.90	2.82
1898.							
Jan.....	39.2	2.90	.....	0.90	0.81	0.81	2.09
Feb.....	37.5	0.62	.....	0.40	0.36	0.36	0.26
Mar	50.8	3.47	.....	1.48	1.33	1.33	2.14
Apr.....	52.9	1.96	0.4	0.99	0.89	1.29	0.67
May	67.4	6.14	1.2	2.94	2.64	3.84	2.30
June ...	73.9	2.82	1.4	1.81	1.63	3.03	—0.21
July....	77.4	4.32	1.6	2.58	2.32	3.92	0.40
Aug	77.2	3.59	1.3	2.11	1.90	3.20	0.39
Sept....	71.2	3.75	0.8	2.00	1.80	2.60	1.15
Oct.....	58.8	6.32	0.3	2.40	2.16	2.46	3.86
	.....	40.74	7.0	19.07	17.16	24.16	16.58

rainfall, the actual variation in inches of rainfall and run-off is practically the same for the Tohickon Creek water-shed, but the variation in inches of rainfall is very much greater than the variation in inches of run-off on the Mississippi River water-shed. This difference exists on all water-sheds having similar differences in annual rainfall and evaporation and transpiration losses. On the Tohickon Creek water-shed, the normal annual rainfall is sufficient to supply the needs of evaporation and transpiration; consequently,

TABLE 25.—SUMMARY OF DATA AND COMPUTATIONS FOR THE TOMBIGBEE RIVER WATER-SHED, ABOVE COLUMBUS, MISS.

Area = 4 440 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1901.....	51.1	24.5	7.2	31.7	19.4	0.4	19.0	19.6
1902.....	45.0	20.4	7.4	27.8	17.2	—0.4	17.6	17.3
1903.....	50.5	21.6	7.1	28.7	21.8	—0.8	22.6	23.1
1904.....	33.6	17.5	8.3	25.8	7.8	0.3	7.5	5.2
1905.....	52.4	23.6	9.0	32.6	19.8	0.6	19.2	15.7‡
1906.....	49.6	24.0	9.3	33.3	16.3	1.7†	14.6	12.8
1907.....	50.3	23.6	9.5	33.1	17.2	2.2†	19.4	18.2
1908.....	54.4	25.2	9.4	34.6	19.8	0.3	19.5	17.8
1909.....	56.2	25.2	8.7	33.9	22.3	—0.3	22.6	24.1
Total.....	443.1	205.6	75.9	281.5	161.6	.....	162.0	153.8
Mean.....	49.2	22.8	8.4	31.2	18.0	.....	18.0	17.1

* From October 1st of previous year to September 30th of given year.

† 6.8 in. of rainfall in September, 1906. 1.5 in. assumed as held over into following year.

‡ 7 months estimated on basis of run-off at Epes.

speaking in very approximate terms, most of the rainfall in addition to those needs appears as run-off, as has been pointed out frequently in the past. On the Mississippi River water-shed, however, and throughout the greater part of the United States, the normal rainfall is insufficient to supply the needs of transpiration and evaporation at the prevailing temperatures; consequently, a large portion of any increased rainfall goes to supply unsatisfied needs of transpiration and evaporation, and hence a comparatively small portion of the increased rainfall, within certain limits, appears as run-off.

Sargent* comments briefly on long-term variations in stream flow on the Croton and Hudson Rivers. It appears from these records, in so far as low water is concerned, that the rate of flow for the 5 dryest months at Mechanicsville, on the Hudson River, was lowest in 1908 and highest in 1905. It was about one-third as much during the former year as during the latter. It also appears that the rate of flow which occurred 70% of the time during the 5 years, 1909 to 1913, was only

* *Engineering News*, December 3d, 1914.

a little more than one-half of that which occurred 70% of the time during the 26 years from 1888 to 1913, even though the extreme minimum rate of flow was practically the same in the two periods.

Fig. 36 shows the progressive 5-year mean rainfall and run-off for the Mississippi River and Tohickon Creek water-sheds. These curves bring out forcibly the great differences which exist, particularly in run-off, between the average values derived from short-term—that is, in this case, 5-year—records.

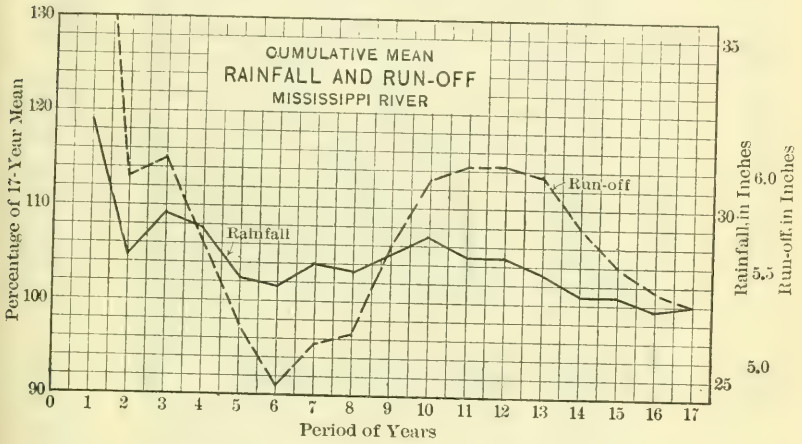


FIG. 34.

If the conclusion as to mean annual run-off for the Mississippi River water-shed were based on the 5-year mean ending in 1902, during which period the rainfall averaged 98% of the mean for the 17-year period, this conclusion would be 20% too low. If the conclusion were based on the 5-year mean ending in 1909, during which period the rainfall was 104% of the mean for the 17-year period, the figure would be 40% in error. If the conclusion were based on the 5-year period ending in 1913, during which period the rainfall was about 10% below normal, the value adopted would be nearly 35% too small. The maximum variation in 5-year means of run-off within the 17-year period over which the records used in this paper extend, is about 75 per cent.

Even though, on a small water-shed such as that of Tohickon Creek, the fluctuations are not as great as they are on the Mississippi, nevertheless, very substantial differences exist between the 5-year mean rainfall and run-off and the 24-year mean.

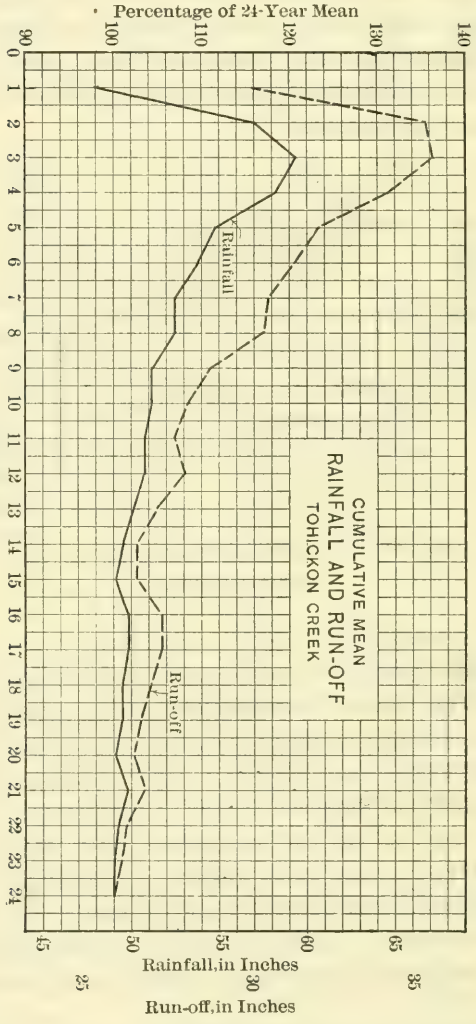

Percentage of 24-Year Mean
90
100
110
120
130
140
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
Period of Years
Rainfall
Run-off
CUMULATIVE MEAN
RAINFALL AND RUN-OFF
TOHICKON CREEK
45
50
55
60
65
Rainfall, in Inches
25
30
35
Run-off, in Inches

FIG. 35.

TABLE 26.—Data and Computations for the Tombigbee River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1905.							
Oct.....	63	4.5	0.6	1.9	2.00	2.60	1.90
Nov....	53	2.0	0.3	0.8	0.84	1.14	0.86
Dec.....	40	5.7	.....	1.4	1.47	1.47	4.23
1906.							
Jan.....	44	3.7	.....	1.3	1.37	1.37	2.33
Feb.....	40	1.7	.....	0.8	0.84	0.84	0.86
Mar	47	5.9	0.2	2.0	2.10	2.30	3.60
Apr	64	1.6	1.1	1.0	1.05	2.15	—0.55
May....	68	4.9	1.5	2.6	2.73	4.23	0.67
June ...	78	2.5	1.6	1.7	1.79	3.39	—0.89
July	78	7.8	1.8	4.1	4.30	6.10	1.70
Aug	80	2.5	1.2	1.7	1.79	2.99	—0.49
Sept....	78	6.8	1.0	3.5	3.68	4.68	2.12
	.....	49.6	9.3	22.8	23.96	33.26	16.34
Oct.....	60	3.8	0.6	1.6	1.68	2.28	1.52
Nov....	53	3.9	0.3	1.4	1.47	1.77	2.13
Dec.....	47	4.4	0.1	1.4	1.47	1.57	2.83
1907.							
Jan.....	51	2.1	0.1	1.0	1.05	1.15	0.95
Feb.....	45	5.2	0.1	1.7	1.79	1.89	3.31
Mar	63	3.9	0.6	1.9	2.00	2.60	1.30
Apr	55	5.1	1.1	2.1	2.21	3.31	1.79
May....	65	10.3	1.6	4.0	4.20	5.80	4.50
June ...	75	2.3	1.5	1.5	1.58	3.08	—0.78
July....	81	5.0	1.7	3.0	3.15	4.85	0.15
Aug	82	1.8	1.0	1.4	1.47	2.47	—0.67
Sept....	74	2.5	0.8	1.5	1.58	2.38	0.12
	.....	50.3	9.5	22.5	23.65	33.15	17.15

In northern latitudes, where practically all the winter precipitation occurs as snow, and streams are fed from ground and lake storage, and where back-water conditions, as a rule, make the open-water rating curve inapplicable during the winter, methods of computing run-off also find application. Their principal use, however, lies in extending observed run-off records so as to include more nearly the extremes of discharge, and to give a more accurate value for mean utilizable flow, than can possibly be obtained from short-term records.

TABLE 27.—SUMMARY OF DATA AND COMPUTATIONS FOR THE COLORADO RIVER WATER-SHED, ABOVE AUSTIN, TEX.

Area = 37 000 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1900	37.4	23.9	10.9	34.8	2.6	0	2.6	0.62
1901	15.7	10.7	4.3	15.0	0.7	0	0.7	0.95
1902	34.0	21.4	8.8	30.2	3.8	2.5	1.3	0.62
1903	21.1	14.8	8.6	23.4	—2.3	—2.5	0.2	0.75
1904	24.4	16.1	7.5	23.6	0.8	0.5	0.3	0.70
1905	31.7	21.1	10.0	31.1	0.6	—0.8	1.4	1.14
1906	25.6	18.3	7.1	25.4	0.2	0	0.2	0.68
1907	28.1	17.3	7.9	25.2	2.9	0.4	2.5	0.98
1908	29.2	19.1	10.1	29.2	0.0	—0.2	0.2	0.62
1909	21.6	14.6	5.7	20.3	1.3	0.1	1.2	0.30
Total.	268.8	177.3	80.9	258.2	10.6	.....	10.6	7.36
Mean.	26.9	17.7	8.1	25.8	1.1	.....	1.06	0.74

*From March 1st of given year to February 28th or 29th of following year.

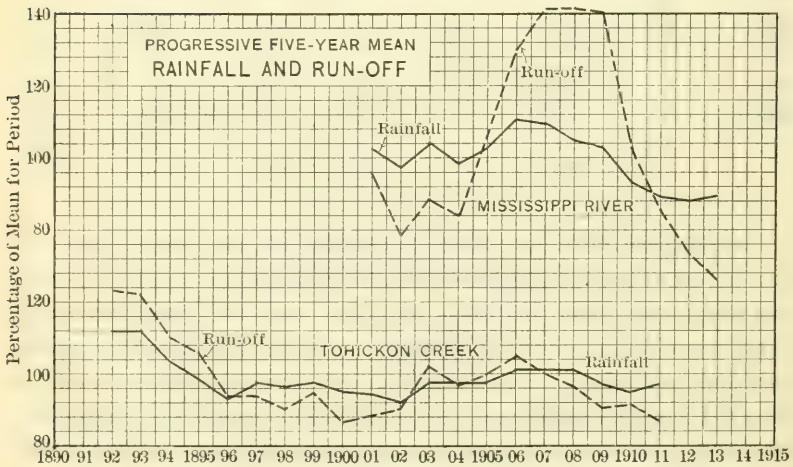


FIG. 36.

TABLE 28.—Data and Computations for the Colorado River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1900.							
Mar.....	56.9	3.38	0.4	1.60	1.92	2.32	1.06
Apr.....	64.8	7.63	1.2	3.35	4.02	5.22	2.41
May.....	72.4	5.47	1.7	2.85	3.42	5.12	0.35
June....	82.0	0.42	1.3	0.45	0.54	1.84	—1.42
July.....	80.6	3.97	1.5	2.51	3.02	4.52	—0.55
Aug	79.8	3.12	1.2	2.00	2.40	3.60	—0.48
Sept....	79.3	6.40	1.6	3.45	4.13	5.73	0.67
Oct.....	69.0	3.17	1.0	1.75	2.10	3.10	0.07
Nov.....	57.3	1.10	0.5	0.55	0.66	1.16	—0.06
Dec.....	48.4	1.07	0.2	0.41	0.49	0.69	0.38
1901.							
Jan.....	56.8	0.25	0.2	0.31	0.37	0.57	—0.32
Feb.....	46.0	1.45	0.1	0.71	0.85	0.95	0.50
	.....	37.43	10.9	19.94	23.92	34.82	2.61
Mar.....	56.2	0.90	0.3	0.60	0.72	1.02	—0.12
Apr.....	62.1	1.40	0.6	*0.70	0.84	1.44	—0.04
May.....	72.6	3.18	0.9	*1.76	2.11	3.01	0.17
June....	82.3	0.52	0.4	*0.35	0.42	0.82	—0.30
July....	84.8	1.88	0.4	*1.40	1.68	2.08	—0.20
Aug....	84.8	1.10	0.2	*0.85	1.02	1.22	—0.12
Sept....	76.8	2.52	0.7	*1.50	1.80	2.50	0.02
Oct.....	69.7	0.88	0.2	*0.50	0.60	0.80	0.08
Nov.....	57.9	1.62	0.5	*0.65	0.78	1.28	0.34
Dec.....	46.9	0.45	0.1	*0.15	0.18	0.28	0.17
1902.							
Jan.....	44.8	0.57	0.0	0.21	0.25	0.25	0.32
Feb.....	47.2	0.64	0.0	0.26	0.31	0.31	0.33
	.....	15.66	4.3	8.93	10.71	15.01	0.65

* Low storage.

TABLE 29.—Summary of Data and Computations for the Sacramento River Water-shed, Above Red Bluff, Cal.

Area = 10 400 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1903	28.3	8.3	2.2	10.5	17.8	17.60
1904	38.9	9.1	2.0	11.1	27.8	28.34
1905	37.8	12.0	3.7	15.7	22.1	19.23
1906	33.5	9.5	3.0	12.5	21.0	19.92
1907	35.7	8.8	1.7	10.5	25.2	24.67
1908	25.3	7.5	1.8	9.3	16.0	14.04
1909	37.5	7.3	1.7	9.0	28.5	25.82
1910	23.1	6.9	2.3	9.2	13.9	16.14
1911	29.6	7.2	3.0	10.2	19.4	17.84
Total	289.7	76.6	21.4	98.0	191.8	183.60
Mean	32.2	8.5	2.4	10.9	21.3	20.4

* September 1st of previous year to August 31st of given year.

Note.—A water year from September 1st to August 31st was used for this water-shed in order to permit the highest possible equalization of storage. September of practically every year marks a decided increase in precipitation after several months of distinctly dry weather.

TABLE 30.—DATA AND COMPUTATIONS FOR THE SACRAMENTO RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1903.							
Sept	61	0.2	0.1	0.20	0.17	0.27	—0.07
Oct	55	1.4	0.3	0.65	0.55	0.85	0.55
Nov	44	8.0	0.2	1.95	1.66	1.86	6.14
Dec	38	2.5	.....	0.65	0.55	0.55	1.95
1904.							
Jan	35	2.0	.....	0.80	0.68	0.68	1.32
Feb	40	9.0	.....	1.97	1.68	1.68	7.32
Mar	38	11.0	.....	1.80	1.53	1.53	9.47
Apr	48	3.8	0.4	1.48	1.26	1.66	2.14
May	58	0.6	0.5	0.50	0.42	0.92	—0.32
June	66	0.1	0.3	0.27	0.23	0.53	—0.43
July	73	0.3	0.2	0.37	0.31	0.51	—0.21
Aug	74	0.0	0.0	0.11	0.09	0.09	—0.09
	.....	38.9	2.0	10.75	9.13	11.13	27.79
Sept	66	3.2	0.1	1.63	1.39	1.49	1.71
Oct	55	5.5	0.3	1.96	1.67	1.97	3.53
Nov	48	4.3	0.1	1.35	1.15	1.25	3.05
Dec	37	3.7	.....	0.89	0.76	0.76	2.94
1905.							
Jan	41	6.0	0.1	1.75	1.49	1.59	4.41
Feb	44	3.0	0.3	1.15	0.98	1.28	1.72
Mar	47	7.0	0.5	2.29	1.95	2.45	4.55
Apr	54	2.0	0.8	1.00	0.85	1.65	0.35
May	55	2.5	0.9	1.25	1.07	1.97	0.53
June	67	0.6	0.5	0.50	0.42	0.92	—0.32
July	77	0.0	0.1	0.19	0.16	0.26	—0.26
Aug	75	0.0	.....	0.11	0.09	0.09	—0.09
	.....	37.8	3.7	14.07	11.98	15.68	22.12

TABLE 31.—Summary of Data and Computations for the Pit River Water-shed, Above Bieber, Cal.

Area = 2 950 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1904.....	19.4	8.2	3.2	11.4	8.0	....
1905.....	11.7	6.2	3.3	9.5	2.2	2.00
1906.....	15.1	6.8	3.0	9.8	5.3	4.73
1907.....	17.4	7.6	3.1	10.7	6.7	7.70
1908.....	9.8	5.9	2.6	8.5	1.3	1.23
1909.....	15.2	6.7	2.6	9.3	5.9	....
Total.....	88.6	41.4	17.8	59.2	29.4	15.66
Mean...	14.8	6.9	3.0	9.9	4.9 †(3.87)	3.92

*From September 1st of previous year to August 31st of given year.
†Mean of years over which observations extend.

TABLE 32.—Data and Computations for the Pit River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1906.							
Sept.	60	0.58	0.2	0.32	0.35	0.55	0.03
Oct.	49	0.18	0.1	0.14	0.15	0.25	—0.07
Nov.	36	1.24	.....	0.30	0.33	0.33	0.91
Dec.	34	2.72	.....	0.60	0.66	0.66	2.06
1907.							
Jan.	25	1.68	.....	0.65	0.71	0.71	0.97
Feb.	42	3.25	0.1	1.18	1.30	1.40	1.85
Mar.	35	3.72	0.1	1.11	1.22	1.32	2.40
Apr.	48	0.41	0.4	0.42	0.46	0.86	—0.45
May.	52	1.26	0.8	0.70	0.77	1.57	—0.31
June.	55	1.84	0.9	0.95	1.05	1.95	—0.11
July.	67	0.28	0.3	0.35	0.38	0.68	—0.40
Aug.	63	0.23	0.2	0.21	0.23	0.43	—0.20
	.....	17.39	3.1	6.93	7.61	10.71	6.68
Sept.	57	1.10	0.3	0.54	0.59	0.89	0.21
Oct.	54	1.62	0.4	0.75	0.82	1.22	0.40
Nov.	38	0.91	.....	0.26	0.29	0.29	0.62
Dec.	35	1.90	.....	0.44	0.48	0.48	1.42
1908.							
Jan.	34	1.05	.....	0.62	0.68	0.68	0.37
Feb.	35	0.34	.....	0.45	0.49	0.49	—0.15
Mar.	37	0.40	.....	0.45	0.49	0.49	—0.09
Apr.	48	0.23	0.1	0.30	0.33	0.43	—0.20
May.	47	1.36	0.5	0.70	0.77	1.27	0.09
June.	57	0.64	0.7	0.48	0.53	1.23	—0.59
July.	72	0.16	0.4	0.25	0.27	0.67	—0.51
Aug.	67	0.10	0.2	0.15	0.17	0.37	—0.27
	.....	9.81	2.6	5.39	5.91	8.51	1.30

TABLE 33.—SUMMARY OF DATA AND COMPUTATIONS FOR THE MCCLOUD RIVER WATER-SHED, ABOVE GREGORY, CAL.

Area = 608 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1903	53.2	6.9	1.7	8.6	44.6	47.80
1904	81.9	8.2	2.2	10.4	71.5	73.98
1905	62.5	10.6	2.6	13.2	49.3	52.10
1906	60.6	9.1	2.6	11.7	48.9	51.27
1907	66.0	7.8	2.7	10.5	55.5	59.87
1908	47.4	6.8	2.4	9.2	38.2	39.05
Total	371.6	49.4	14.2	63.6	308.0	324.07
Mean	61.9	8.2	2.4	10.6	51.3	54.0

* From September 1st of previous year to August 31st of given year.

NOTE.—A water year from September 1st to August 31st was used for this water-shed in order to permit the highest possible equalization of storage. September of practically every year marks a decided increase in precipitation after several months of markedly dry weather.

TABLE 34.—Data and Computations for the McCloud River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1903.							
Sept....	62	0	0.1	0.10	0.06	0.16	—0.16
Oct.....	58	4.0	0.3	1.60	0.96	1.26	2.74
Nov.....	47	14.0	0.2	2.20	1.32	1.52	12.48
Dec.....	42	4.0	0.1	1.10	0.66	0.76	3.24
1904.							
Jan.....	39	4.5	.....	1.10	0.66	0.66	3.84
Feb.....	42	20.0	.....	2.20	1.32	1.32	18.68
Mar.....	40	28.0	.....	2.00	1.20	1.20	26.80
Apr.....	50	7.0	0.4	2.40	1.44	1.84	5.16
May.....	60	0.4	0.4	0.40	0.24	0.64	—0.24
June ...	68	0	0.3	0.20	0.12	0.42	—0.42
July....	73	0	0.2	0.20	0.12	0.32	—0.32
Aug.....	74	0	0.2	0.10	0.06	0.26	—0.26
	.....	81.9	2.2	13.60	8.16	10.36	71.54
Sept....	67	3.0	0.2	1.60	0.96	1.16	1.84
Oct.....	56	10.0	0.2	2.10	1.26	1.46	8.54
Nov.....	50	4.0	0.2	1.30	0.78	0.98	3.02
Dec.....	40	6.0	.....	1.40	0.84	0.84	5.16
1905.							
Jan.....	43	12.0	.....	2.20	1.32	1.32	10.68
Feb.....	45	7.0	0.1	2.20	1.32	1.42	5.58
Mar.....	48	12.0	0.2	2.70	1.62	1.82	10.18
Apr.....	53	4.0	0.4	1.70	1.02	1.42	2.58
May.....	54	4.0	0.5	1.70	1.02	1.52	2.48
June....	65	0.5	0.5	0.40	0.24	0.74	—0.24
July....	76	0	0.2	0.20	0.12	0.32	—0.32
Aug.....	73	0	0.1	0.10	0.06	0.16	—0.16
	.....	62.5	2.6	17.60	10.56	13.16	49.34

TABLE 35.—(*Continued.*)

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Transpiration, in inches.	Evaporation, in Inches.		Total loss, in inches.	Precipitation minus total loss, in inches.	Computed Run-off, in Inches, over Water-shed.			Observed Run-off.		Storage over Water-shed, in Inches.					Notes.
				From curve.	Actual.			Surface flow.	Seepage flow.	Total.	At Houston.	At Lanesboro.	Monthly storage.	Cumulative storage on first of month.				
														Surface.	Soil.	Subsoil.	Total.	
1911.																		
Mar....	35	0.8	.....	0.40	0.49	0.61	0.31	0.15	0.14	0.29	0.29	0.24	0.02	0.50	1.30	3.56	5.36	(9)
Apr....	45	1.4	0.3	0.70	0.86	1.16	0.24	0.05	0.17	0.22	0.25	0.22	0.02	.....	1.50	3.88	5.38	(10)
May....	64	5.4	1.6	2.50	3.06	4.66	0.74	0.15	0.15	0.30	0.32	0.33	0.44	.....	1.70	3.70	5.40	(11)
June...	73	3.2	1.7	1.95	2.39	4.09	−0.89	0.10	0.14	0.24	0.27	0.21	−1.13	.....	2.30	3.54	5.84	
July....	72	3.2	1.7	1.90	2.33	4.03	−0.83	.....	0.12	0.12	0.21	0.11	−0.95	.....	1.40	3.31	4.71	
Aug....	68	6.0	2.1	*2.60	3.18	5.28	0.72	0.20	0.11	0.31	0.81	1.76	0.41	.....	0.60	3.16	3.76	(12)
Sept....	61	4.3	1.5	1.75	2.14	3.64	0.66	0.15	0.10	0.25	0.34	0.31	0.41	.....	1.00	3.17	4.17	(13)
Oct.....	46	7.4	0.3	2.00	2.45	2.75	4.65	1.40	0.10	1.50	1.03	1.78	3.15	.....	1.50	3.08	4.58	(14)
Nov....	25	1.9	.....	0.40	0.49	0.49	1.41	0.30	0.20	0.50	0.38	0.47	0.91	.....	3.50	4.23	7.73	(15)
Dec.....	25	2.8	.....	0.50	0.61	0.61	2.19	0.60	0.29	0.89	0.65	0.82	1.30	0.70	3.00	4.94	8.64	(16)
1912.																		
Jan.....	−3	0.7	.....	0.05	0.06	0.06	0.64	.....	0.35	0.35	0.36	0.32	0.29	1.50	3.00	5.44	9.94	
Feb....	14	0.4	.....	0.30	0.37	0.37	0.03	.....	0.31	0.31	0.28	0.25	−0.28	2.10	3.00	5.13	10.23	
										5.28	5.19							
Mar....	23	0.9	.....	0.50	0.61	0.61	0.29	0.60	0.27	0.87	0.92	1.46	−0.58	2.10	3.00	4.85	9.95	(17)
Apr.....	49	2.9	0.4	1.20	1.47	1.87	1.03	0.80	0.34	1.14	1.12	1.54	−0.11	1.00	3.00	5.37	9.37	(18)
May....	59	4.9	1.5	2.15	2.64	4.14	0.76	0.40	0.48	0.88	0.78	1.07	−0.12	.....	3.00	6.26	9.26	(19)
June...	64	2.0	1.8	1.20	1.47	3.27	−1.27	.....	0.45	0.45	0.40	0.40	−1.72	.....	3.00	6.14	9.14	
July....	71	6.0	2.2	3.00	3.68	5.88	0.12	0.05	0.39	0.44	0.61	0.52	−0.32	.....	1.70	5.72	7.42	(20)
Aug....	67	3.5	2.0	1.70	2.08	4.08	−0.58	.....	0.34	0.34	0.33	0.30	−0.92	.....	1.70	5.40	7.10	(21)
Sept....	61	2.0	1.2	1.00	1.22	2.42	−0.42	.....	0.30	0.30	0.30	0.28	−0.72	.....	1.10	5.08	6.18	(22)
Oct.....	50	1.9	0.4	0.70	0.86	1.26	0.64	0.10	0.26	0.36	0.31	0.29	0.28	.....	0.70	4.76	5.46	
Nov....	37	1.4	.....	0.35	0.43	0.43	0.97	0.10	0.24	0.34	0.29	0.28	0.63	.....	1.20	4.54	5.74	
Dec.....	25	1.7	.....	0.40	0.49	0.49	1.21	0.10	0.22	0.32	0.24	.....	0.89	.....	2.00	4.37	6.37	(23)
1913.																		
Jan.....	16	0.5	.....	0.40	0.49	0.49	0.01	.....	0.20	0.20	0.20	0.18	−0.19	0.50	2.60	4.16	7.26	
Feb.....	14	0.8	.....	0.35	0.43	0.43	0.37	.....	0.18	0.18	0.18	0.17	0.19	0.50	2.60	3.97	7.07	
										5.82	5.68							

* Most of rain fell in one day—evaporation reduced about 0.1 in.

(9) Maximum temperature on one day 76 degrees.
(10) Last days of March warm, some run-off carried forward into April; rain well distributed during April.
(11) About ½ in. of rain, May 31st and about 3 in. of rain, June 2d-4th.
(12) Most of rain in one day.
(13) Rain well distributed.
(14) Rain well distributed.
(15) Month cold; precipitation very well distributed.
(16) About 1 in. of rain on 10th.
(17) Month cold, except last week, which was quite warm.
(18) Rain well distributed.
(19) Rain very well distributed.
(20) Considerable rain on 9th-11th.
(21) Rain very well distributed.
(22) Rain very well distributed.
(23) About ½ in. of rain.

TABLE 35.—Data and Computations for the Root River Water-shed.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Transpiration, in inches.	Evaporation, in Inches.		Total loss, in inches.	Precipitation minus total loss, in inches.	Computed Run-off, in Inches, over Water-shed.			Observed Run-off.		Storage over Water-shed, in Inches.					Notes.
				From curve.	Actual.			Surface flow.	Seepage flow.	Total.	At Houston.	At Lanesboro.	Monthly storage.	Cumulative storage on first of month. Surface.	Soil.	Subsoil.	Total.	
1909.																		
June...	67	3.6	2.0	1.90	2.33	4.33	—0.73	.....	0.45	0.45	0.45	.....	—1.18	.....	2.00	6.10	8.10	
July....	71	0.7	1.3	0.60	0.74	2.04	—1.34	.....	0.38	0.38	0.33	.....	—1.72	.....	1.30	5.62	6.92	
Aug....	74	7.9	2.4	3.75	4.60	7.00	0.90	0.30	0.32	0.62	0.69	.....	0.28	.....	0.00	5.20	5.20	(1)
Sept....	60	4.5	1.6	1.85	2.27	3.87	0.63	0.10	0.28	0.38	0.37	.....	0.25	.....	0.60	4.88	5.48	(2)
Oct.....	47	1.3	0.3	0.50	0.61	0.91	0.39	0.10	0.25	0.35	0.32	.....	0.04	.....	1.10	4.63	5.73	
Nov.....	39	6.5	.....	1.50	1.84	1.84	4.66	0.80	0.21	1.01	0.83	.....	3.65	.....	1.50	4.27	5.77	(3)
Dec.....	15	1.5	.....	0.30	0.37	0.37	1.13	0.20	0.22	0.42	0.44	.....	0.71	2.00	3.00	4.42	9.42	(4)
1910.																		
Jan.....	13	1.7	.....	0.30	0.37	0.37	1.33	.....	.....	0.38	0.37	.....	0.95	2.50	2.00	5.63	10.13	
Feb.....	12	0.3	.....	0.30	0.37	0.37	—0.07	.....	.....	0.33	0.30	0.24	—0.40	3.80	2.00	5.28	11.08	
										4.32	4.10							
Mar....	45	0.0	0.2	0.30	0.25	0.45	—0.45	0.70	0.29	0.99	0.83	1.48	—1.44	3.70	2.00	4.98	10.68	(5)
Apr....	51	1.2	0.8	0.60	0.74	1.54	—0.34	0.10	0.40	0.50	0.40	1.04	—0.84	0.50	3.00	5.74	9.24	
May...	53	2.6	1.3	1.10	1.35	2.65	—0.05	.....	.....	0.42	0.37	0.38	—0.47	.....	2.50	5.90	8.40	
June...	70	0.2	1.5	0.20	0.25	1.75	—1.55	.....	.....	0.36	0.28	0.28	—1.91	.....	2.50	5.53	8.03	
July....	75	0.9	1.0	0.65	0.80	1.80	—0.90	.....	.....	0.31	0.24	0.24	—1.21	.....	1.00	5.12	6.12	
Aug....	70	4.1	1.4	2.10	2.57	3.97	0.13	.....	.....	0.27	0.25	0.32	—0.14	.....	0.10	4.81	4.91	(6)
Sept....	60	3.5	1.2	1.50	1.84	3.04	0.46	0.05	0.24	0.29	0.33	0.40	0.17	.....	0.20	4.57	4.77	
Oct.....	53	0.7	0.3	0.35	0.43	0.73	—0.03	.....	0.22	0.22	0.25	0.26	—0.25	.....	0.60	4.34	4.94	
Nov....	29	0.4	.....	0.10	0.12	0.12	0.28	0.05	0.20	0.25	0.25	0.29	0.03	.....	0.50	4.19	4.69	
Dec.....	19	0.5	.....	0.25	0.31	0.31	0.19	.....	0.18	0.18	0.23	.....	0.01	.....	0.70	4.02	4.72	
1911.																		
Jan.....	13	0.9	.....	0.30	0.37	0.37	0.53	0.05	0.16	0.21	0.23	.....	0.32	0.20	0.70	3.83	4.73	(7)
Feb....	23	1.5	.....	0.60	0.74	0.74	0.76	0.30	0.15	0.45	0.56	*0.20	0.31	0.70	0.70	3.65	5.05	(8)
										4.45	4.22							

(1) 6½ in. of rain in 5 days.
(2) 4 in. of rain in 3 days.
(3) About 2 in. of snow, last week quite warm.
(4) Some snow melted during first week of month.
(5) High temperature, latter part of month.
(6) Heavy rain Aug. 31st and Sept. 5th.
(7) A few warm days, 24th-25th.
(8) 12th-17th warm, about 1 in. of rain.

TABLE 35.—(*Continued.*)

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Transpiration, in inches.	Evaporation, in Inches.		Total loss, in inches.	Precipitation minus total loss, in inches	Computed Run-off, in Inches, over Water-shed.			Observed Run-off.		Storage over Water-shed, in Inches.					Notes.
				From curve.	Actual.			Surface flow.	Seepage flow.	Total.	At Houston.	At Lanesboro.	Monthly storage.	Cumulative storage on first of month. Surface.	Soil.	Subsoil.	Total.	
1913.																		
Mar.....	27	3.2	.....	0.90	1.10	1.10	2.10	1.50	0.16	1.66	*2.48	2.27	0.44	0.90	2.60	3.76	7.26	(24)
Apr.....	46	1.5	0.3	0.75	0.92	1.22	0.28	0.20	0.26	0.46	0.46	.....	—0.18	.....	3.00	4.70	7.70	.....
May	56	5.4	1.3	2.25	2.75	4.05	1.35	0.25	0.30	0.55	0.38	.....	0.80	.....	2.50	5.02	7.52	(25)
June	72	3.2	2.0	1.90	2.33	4.33	—1.13	.....	0.34	0.34	0.28	.....	—1.47	.....	3.00	5.32	8.32	.....
July	72	5.3	2.1	2.80	3.43	5.53	—0.23	0.10	0.30	0.40	0.42	.....	—0.63	.....	1.80	5.05	6.85	(26)
Aug.....	72	2.6	1.7	1.50	1.84	3.54	—0.94	.....	0.27	0.27	0.30	.....	—1.21	.....	1.40	4.82	6.22	.....
Sept	62	3.7	1.4	1.65	2.02	3.42	0.28	.....	0.25	0.25	0.24	.....	0.03	.....	0.40	4.61	5.01	(27)
Oct.....	47	2.6	0.3	0.85	1.05	1.35	1.25	0.10	0.20	0.30	0.28	.....	0.95	.....	0.70	4.24	4.94	(28)
Nov.....	41	1.7	.....	0.50	0.61	0.61	1.09	0.10	0.19	0.29	0.25	.....	0.80	.....	1.80	4.09	5.89	(29)
Dec.....	31	0.2	.....	0.08	0.10	0.10	0.10	.....	0.20	0.20	0.26	.....	—0.10	.....	2.50	4.19	6.69	.....
1914.																		
Jan.....	24	1.4	.....	0.50	0.61	0.61	0.79	.....	0.19	0.19	0.22	.....	0.60	.....	2.50	4.09	6.59	.....
Feb.....	11	0.5	.....	0.20	0.20	0.24	0.26	.....	0.17	0.17	0.17	.....	0.09	0.80	2.50	3.89	7.19	.....
										5.08	5 74						

(24) The month of destructive storms, minimum temperature, 10th–15th above freezing—heavy rain on 14th.
(25) Precipitation well distributed—no heavy downpours.
(26) Rather heavy rains 4th–5th.
(27) Rain well distributed.
(28) Rain well distributed.
(29) No snow—rain well distributed.
* Estimated from Lanesboro.

NOTE.—The 1913 run-off figures are advance estimates furnished through the courtesy of the District Office, U. S. G. S., and are subject to revision for publication. Official data will probably be available very soon.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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EXTERNAL CORROSION OF CAST-IRON PIPE
Discussion.*

By Marshall R. Pugh, M. Am. Soc. C. E.

Mr. Pugh.

Marshall R. Pugh,† M. Am. Soc. C. E. (by letter).—In a number of instances corrosion of steel pipe is closely linked with that of cast-iron pipe, is due to the same causes, and is to be prevented by similar methods. The conditions cited by Mr. Bowie in the California oil pipe lines are severe. The high pressure to which they are subjected and, still more, the great temperature variations, undoubtedly call for special treatment. The writer was much interested in the method of wrapping, which is quite similar to that recommended for protecting the Goldfields main in Western Australia.‡ In this case a 30-in. steel pumping main, 350 miles long, used to supply Coolgardie, was found to be rapidly failing from both internal and external corrosion. As the cost of the main was about $10 000 000, it was a matter of very great importance. It appeared on investigation that two of the recommendations of the commission of engineers had not been followed in its construction; first, that the pipe should be dipped in a nearly boiling asphaltic composition, both in England and immediately before being laid in Australia; and second, that the pipe be laid above ground. In reference to the first point, the Joint Report says:

"(1) If the pipes are dipped just before laying, the deterioration of the coating due to exposure or possible damage in transmit is minimised. (2) It has been found that a coating which appears perfectly satisfactory to the naked eye contains, when examined with a magnifying glass, small holes due to minute bubbles which penetrate right

* Author's closure of the paper by Marshall R. Pugh, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Philadelphia, Pa.

‡ "Goldfields Water Supply, Corrosion of Steel Main." Joint Report of Sir Alexander Binnie, Son and Deacon, Sir William Ramsay, F. R. S., and Mr. Otto Hehner, F. C. S.

Mr. Pugh. through to the metal beneath. On removing one of the nodular incrustations which form, as a result of corrosion, the coating at first sight frequently appears to be sound. A more minute examination, however, reveals the fact that the metal has decayed below the coating, having been attacked by the penetration of the water through one of these minute holes to the metal surface. Were the pipe to be dipped a second time as recommended by the Commission, care being taken that the first coat remained uninjured, the odds against the occurrence of two air bubbles, one above the other, giving direct access of the water to the metal, would be enormous."

The remedial measures recommended have been stated in the discussion by Mr. W. J. E. Binnie.

The writer would be interested in knowing the cost of wrapping pipe with roofing papers coated with asphaltum, and how it compares with the cost of using galvanized pipe.

The cast-iron saddle shown by Mr. Bowie, Fig. 26, is analogous to the more primitive expedient on the 30-in. steel main at Atlantic City, where the hole was closed with a wooden plug, and a box was strapped to the pipe and filled with concrete.

The expansion and contraction due to temperature variations in the oil pipe lines obviously makes concrete a poor material with which to protect them, but if surrounded by a lime and clay puddle, the desired result might be obtained. Whenever pipes are cased in concrete it would be of great advantage to adopt the expedient mentioned in the discussion relative to the Birkenhead pipe. In this instance the joints are not surrounded with concrete, but are buried in a puddle of clay and slaked lime. This enables a length of pipe to be removed without the necessity of cutting the concrete.

Mr. Allen, whose opinion should have great weight, estimates the extreme life of cast-iron pipe, under conditions obtaining at Atlantic City, as 25 years.

The question raised by Mr. Purver as to whether or not the rate of rusting remains uniform and varies directly with the time of exposure, is difficult to answer with any data in the writer's possession. It would appear reasonable to assume that it goes on at an accelerating rate, because both electrical and physical conditions would tend to promote more intense corrosion, at all events within certain limits. As bearing on this, the case of internal attack mentioned by Mr. Brush is of interest. He states that, at Far Rockaway, pipe laid from 20 to 30 years ago, and cleaned about 10 years ago, now shows tubercles of a depth and volume equal to those in pipe laid at the same time which had not been cleaned. Possibly this may be due in large part to the fact that a scraped pipe has the surface left in very favorable condition for corrosion, whereas the new pipe is protected for quite a number of years before the attacks can even begin.

Mr. Pugh.

For the purpose of observing the effect of skin resistance, the writer had a series of plates made, 1.5 by 2.0 by about 0.28 in. Some of these were of ordinary cast iron, and some were of iron cast in moulds the facing sand of which had been mixed with chromite, $Fe\ Cr\ O_4$, ground to fine powder. Three of the plain plates and three of the chromated plates were immersed in a glass containing 100 cu. cm. of a 3% solution of sea salt in water, which closely approximates the composition of sea water. The edges of all the plates were filed smooth and paraffined, so as to confine the corrosion to the two faces of the plates. The plain cast iron contained no chromium. Filings of the skin of the chromated plates contained 0.42% of chromium.

The results, given in Table 9, do not indicate any very promising outlook for such methods of protection.

TABLE 9.

No. of sample.	Kind of iron.	Weight of sample.	Weight, 10 days.	Weight, 54 days.	Weight, 5 months.	Loss of weight in 5 months.	Loss of weight per square inch of exposed surface.
1	Plain cast iron.	108.765	108.716	108.600	108.180	0.585	0.0975
2		97.511	.....	.....	96.888	0.623	0.104
3		94.307	.....	.....	93.793	0.514	0.0857
4	Chromated cast iron.	97.421	97.357	97.252	96.874	0.547	0.091
5		101.209	.....	.....	100.561	0.648	0.108
6		100.765	.....	.....	100.084	0.681	0.113

In corroboration of what Mr. Flinn has said about the protective effect of galvanizing on the Catskill Aqueduct work, the writer had an opportunity to ascertain the condition of a galvanized-iron ocean outfall sewer constructed under his supervision 11 years ago. This pipe was recently exposed at mean tide level near the shore, where it had constantly been subjected during that period to alternate action of air and water externally and sewage internally. It was a matter of comment by all who saw it, how bright, new, and uninjured the galvanizing was. The pipe was apparently as good as the day it was laid, and the zinc still retained its bright, fresh appearance.

Mr. Doten has presented an interesting and unique case of cast-iron pipe corrosion, and his experiments would seem to establish the correctness of his conclusion that "the action causing the disintegration of the metal was electro-chemical, and resulted from the contact of the coal (carbon) with the iron in an electrolyte (sea water)." In a general way it is analogous to the instance cited by Krzizan,* and, taken in conjunction with that case, shows need for care in a number

* *Proceedings*, Am. Soc. C. E., for August, 1914, p. 1652.

Mr. Pugh. of what at first sight might appear to be harmless conditions. The writer, nevertheless, cannot go so far as to ascribe all such disintegration to this source. For example, a case was brought to his attention by Mr. B. B. Hodgman, in which a peat bog of great depth, a mile or so from Belle Plaine, Iowa, was crossed by the Chicago and North Western Railway. The Railway Company about 1894 laid a 6-in. cast-iron water pipe, some 1 800 ft. long, from a pumping station to its roundhouse. It passed through this bog, in which it was originally laid from 18 to 20 in. deep, and in 10 years had sunk to a depth of about 4 ft. When examined by Mr. Hodgman in 1908 the pipe had been in place some 14 years. It was badly corroded externally, with pittings ¼ in. deep and 1½ in. in diameter. In this case there was no salt water to act as an electrolyte.

A peat bog is rich in the humus acids; the ligneous fiber of the sphagnum mosses on decaying gives rise by a fermentive process to humic, ulmic, and other rather ill-defined acids. These acids are powerful agents in dissolving mineral matter from rocks, and even quartz may be corroded by them. Large deposits of bog-iron ore are found in such swamps. The corrosion of the Belle Plaine pipe would thus seem to be readily explained.

That coal may be an important contributing cause when cast-iron pipe laid in cinders is subjected to the action of salt water would appear probable, but there is also no question of the dominant rôle played by sulphur, as noted at the Harrisburg tunnel.*

Mr. T. N. Thomson† cites two cases of corrosion in wrought-iron and steel pipes. One, a piece of 1½-in. black wrought-iron exhaust pipe from power boilers in Tonawanda, N. Y., had lain in damp soil and ashes 4 years. No pitting was present, but the pipe varied from about half its original thickness down to that of tissue paper. The second, a 2-in. steel pipe, from Boston, was laid on top of a cement floor over which was a plank floor on which coal was stored. Some of this coal worked down through the planking so that the pipe was partly buried in coal. After each heavy rain, surface water leaked into the coal, so that it is probable the coal around the pipe was constantly moist. This pipe, laid in February, 1904, failed through external corrosion, and had to be removed a year later. It is scarcely safe to assume that this was due entirely to what Mr. Doten terms electro-chemical action. It is a well-known fact that the water leaching through coal becomes strongly acid from the sulphur in the iron pyrites found in the coal. The acid waters, leached from old culm banks in the coal regions, in many cases have destroyed every vestige of vegetable matter near them. These acid waters, as already stated

* *Proceedings*, Am. Soc. C. E., for August, 1914, p. 1685.

† *Journal*, Engrs. Soc. of Pennsylvania, Vol. 1, p. 233.

in the paper, are very destructive of iron, and may have to do with the destruction of the substructure of coal wharves, cited by Mr. Doten. Mr. Pugh.

The writer still adheres to his views regarding the effect of air on the rapidity of corrosion of different parts of the pipe as noted on page 1679* and illustrated by Fig. 22. This explains satisfactorily why, at Atlantic City, the pipe was perforated most frequently at the top, whereas at Elizabeth it deteriorated chiefly at the bottom. Nor is this inconsistent with the fact that pipe laid in open trench, but still subjected to alternate wetting and drying, lasts longer than pipe in a marsh, although that in open trench is exposed more freely to the air whenever the water falls. The moist, aerated soil forms just the condition for most vigorous action, whether chemical or electro-chemical. Furthermore, in the soil, the ferric hydroxide is retained mechanically in contact with the pipe in large quantity, as shown by the analysis in Table 7. Ferric hydroxide acts as a catalytic agent in hastening many chemical reactions, and may thus accelerate the corrosion of the pipe.

Soil conditions, and even substances accidentally present in the soil, as in the case cited by Mr. Doten, add another factor to the complex interplay or corrosive and inhibitive forces affecting the durability of cast iron.

Apparently insignificant factors may produce far-reaching effects. Mr. F. N. Speller,† in studying the corrosion of iron, took two clean iron rods connected by copper wires to a volt meter, and immersed them in salt water. A slight difference of potential was usually shown. If one of the rods was exposed to the air for a few minutes and then replaced in the water, a strong deflection, amounting to 50 milli volts was indicated with the electrode which was aerated as a cathode. On removing the other rod and replacing it after an equal or slightly longer exposure, the current was reversed.

After a prolonged study of the subject Mr. Speller concluded that "the rate of corrosion evidently depends more on the oxygen available than any other factor". Whether the corrosion proceeds uniformly or is concentrated in pits, seemed to him to depend chiefly on the irregular distribution of foreign impurities in contact with the surface, an idea according well with Mr. Doten's theory.

A statement of the writer on page 1647* is open to misinterpretation. In the clause reading "The salt of the ocean is particularly destructive to cast iron", it was not intended to imply that wrought iron or steel is less seriously affected than cast iron. The writer is absolutely in accord with Mr. Doten's views on the superior resisting power of cast iron under such conditions, and has so stated

* *Proceedings*, Am. Soc. C. E., for August, 1914.

† *Journal*, Engrs. Soc. of Pennsylvania, Vol. 1, p. 243.

Mr. Pugh.

on page 1688.* He is glad to be corroborated so strongly in this view by one having such extended experience in this matter.

So far as concerns the main conclusions of the paper, the writer sees no reason to change them. The discussion has been of great aid in considering the merits of different remedies, and may be summarized as follows:

A.—Increasing the Skin Resistance of Cast Iron.—This deserves investigation, but the outlook is not particularly promising.

B.—Utilizing the Protective Influence of Alkalis by Surrounding the Pipe with Lime or Cement Where Such is Practicable.—This method was advised in the Joint Report for the protection of the Coolgardie, Australia, main which passed through corrosive soils, and is to be used in connection with the new Birkenhead, England, supply where the main is laid in salt marsh. It was adopted with success in Boston Harbor. The discussion would seem to bear out this method as having a wide field of usefulness.

C.—Exclusion of Acids, Salt, or Air.—Under this caption would come the device of raising the pipe above the surface. The discussion brought out a successful instance of this in a pipe laid in 1862, and still in service. The double dipping of pipe, as outlined in the Joint Report, is also deserving of careful consideration. Wrapping the pipe is also shown to be capable of wide application.

D.—Galvanizing the Cast-Iron Pipe, Thus Protecting It at the Expense of the Zinc.—What little was brought out under this head in the discussion tends to point to it as being desirable under certain conditions. Strong evidence is furnished by the example mentioned by the writer of its protective effect on a sewer outfall.

The engineer should be sufficiently familiar with conditions under which cast-iron pipe is subject to rapid deterioration to advise when it may be desirable to spend even relatively large sums for external protective measures, as well as to appreciate when the cost of such protection would be incommensurate with the results obtained.

* *Proceedings*, Am. Soc. C. E., for August, 1914.

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PAPERS AND DISCUSSIONS

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THE CLARIFICATION OF SEWAGE BY FINE SCREENS

Discussion.*

By Messrs. George A. Soper and Kenneth Allen.†

Mr. Soper.

George A. Soper,‡ M. Am. Soc. C. E. (by letter).—Credit is due the author of this paper for calling attention to a form of apparatus for the treatment of sewage concerning which, thus far, little notice has been taken in the United States or England. To many persons connected only with the practical side of sewage works, it will be news that such apparatus as these highly developed screens exist; and yet there is nothing especially new or newly discovered about them. They have been seen for years by all American engineers who have made tours of inspection of European sewerage works.

There has been little inclination to use German screens in the United States, partly because it was not certain that they could accomplish satisfactory results under the conditions which obtain here, and partly on account of the fact that the apparatus was mechanical, proprietary, and not capable of convenient, prompt, and economical repair.

Until recently, no German screen has been pushed commercially in the United States, and few have thought it worth while to translate into English more than fragmentary descriptions of it. Mr. Allen has performed a useful service in bringing together a great deal of information which cannot fail to arouse American interest in a very important type of sewage apparatus and lead to discussions, both in and out of this Society, which will broaden the horizon and add to the resources of sewerage engineers.

* Discussion of the paper by Kenneth Allen, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Author's closure.

‡ New York City.

Mr. Soper. It is to be hoped that this paper will not lead to a too enthusiastic view of the functions of screens. The writer has visited many German works, is familiar with the best installations, and is a strong advocate of screens in their proper places; but he thinks that American engineers should regard them with a great deal of conservatism until they have been well tried. The conditions under which they operate in Europe are quite different from those under which they would be compelled to work here. It is unwise to infer that the foreign results which are reported could be duplicated. It is possible that the German screens can do better here than anywhere; but it is far more likely that they will find their best development in adaptations and improvements which American investigators and American ingenuity can devise. These remarks refer particularly to the more complicated types of screens; a few of the simpler forms seem suitable for use in America with but little change.

So far as the writer sees, screens are sufficiently different from other types of sewage apparatus, and have such individual functions, that they are warranted in being placed in a class by themselves. If this view is correct, it is unfair, both to them and to settling basins, to regard the two as though they were competitors; and yet they are competitors, in the sense of being able to improve sewage by the removal of a part of the solid matter; and comparison for the purpose of bringing out their differences is plainly desirable.

Screens are mechanical contrivances which, for proper operation and repair, require constant and skilled attention. If any of their many moving parts gets out of order, the whole screen may become useless. A very small accident may put one out of operation for a long time. In Germany the labor which is employed in connection with sewage disposal works is better and cheaper than that customarily seen in America, and as attendance plays such an important part in the operation of screens, in comparison with settling basins, the latter have an advantage in this country. Settling basins are suitable for installations of large and of small size, but screens can be used only where the plant is sufficiently large to permit of the expense of proper attendance. The material taken out of the sewage by screens must be removed from the plant every day, whereas that from settling basins can remain for long periods. In the case of some screens, the cost of depreciation and repair must be large, but, unfortunately, there are few figures from unprejudiced sources which can be quoted in this particular. It is trifling in comparison with settling basins.

The results of operation, under the best circumstances, are quite different with screens and settling basins, and although some of the tests which have been reported seem to indicate that they may both do about the same work in the removal of organic matter in sewage, a moment's thought will show that they produce different effects on

the sewage. Screens remove particles according to size, settling basins according to weight. Neither approaches the ideal, which is removal according to composition. Screens, however, have some advantage, inasmuch as a large part of the heavy solid matter which goes to the credit of settling basins is not putrescible. On the other hand, much of the material which floats will not decompose rapidly enough to make trouble if left in the sewage, so that the difference between screens and settling basins on this account is not great. Screens have the disadvantage of breaking up some of the very material which it is most desirable to remove, so that it has been claimed that there was more organic matter in solution in the effluent than in the applied sewage in some plants. There is no such action in settling basins, but the latter have a compensating disadvantage in the fact that they keep the sewage on hand several hours longer, thus favoring decomposition. Mr. Soper.

So far as offensiveness in operation is concerned, the advantage appears to be in favor of the screen when scrupulous care can be taken with it. When operated without regard to the production of odors, screens are undoubtedly among the most offensive kinds of apparatus used in sewage treatment. The liquor contained in the screenings is peculiarly offensive.

American practice certainly should not tolerate the handling of screenings as is done at most of the German works. They should be burnt as soon as possible, unless they are to be utilized by conversion into fertilizer at a plant which is operated under skilled direction. In this event they should be stored temporarily and transported from the screen works in closed and tight containers. It is a mistake to suppose that screenings would have a value in America commensurate with the nuisance which they would create. American farmers do not need such material, and when they get it they do not know how to use it.

Chemical analyses should mislead no one to form the idea that the nitrogen is so valuable that it will well repay a farmer to make use of it. He can get the nitrogen which he needs for much less money in far more convenient form by buying it. The writer is of the opinion that the recovery of a part of the manurial ingredients of sewage is both practicable and desirable, but only in those cases where the works are sufficiently large, well managed, and so situated, as to permit the work of recovery to be carried on to advantage.

Screens have an advantage over settling basins in the fact that they can properly be installed in built-up sections of cities, where it would be inadmissible to place settling basins. This is possible by reason of the smaller space which they take up, the less chance of nuisance, and the less objection which is likely to be made to them by property holders in the vicinity. Well-built settling basins, oper-

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ating on modern principles, do not produce a nuisance, according to the writer's observation, when placed where there is perfect freedom of ventilation, but there is grave doubt as to their inoffensiveness otherwise. It has sometimes been proposed to place settling basins beneath the streets of cities, and in closely covered buildings in the midst of thickly occupied business and residence districts. There seem to be too many risks such as those of explosion, of offensiveness, and of popular objection, to make this proceeding wise. On the other hand, it has been shown to be practicable to build screens in compactly settled districts. This is a particularly suitable field for them, if they can be managed in such a way as to avoid nuisance.

Screening is useful according to the location of the screens. The fresher the sewage, the more effective will be the process. To place fine screens at the end of a long sewer, as is done in some of the German plants, is to limit the quantity of material which they can remove, or to make it necessary to provide screens of very great fineness. More than this, the very kind of material which it is most desirable to remove from the sewage becomes in large part broken up by the friction and submersion in long conduits, and resolved into the troublesome colloid state in which it escapes screens. The Germans are well aware of this, and have placed relatively coarse screens in the built-up section of Hamburg and fine screens at the end of the long outfall at Dresden. It is not probable that these plants would operate at all well if their locations were reversed.

The two most promising uses of screens, in the writer's opinion, are in preparing sewage for direct discharge, under suitable conditions, into natural bodies of water which have sufficient capacity to digest it, and in taking from sewage those solids which can be removed in relatively large quantity, when fresh, and which would be broken up and add materially to the cost of disposal at the end of a more or less distant main sewer.

It should not be supposed that screens of any degree of fineness clarify sewage, in the sense of making it clear or reducing the greasy or discoloring effects which are produced when it is discharged into streams or other natural bodies of water. It can lessen the offensive appearance only to the extent of taking out those particles which are separately recognizable as of sewage origin.

The fact is, clarification, like many other terms used in sewage treatment, is a misnomer. This is fairly well understood, and on this ground, the author is justified in using it, although its continued use is certain to lead to a wrong impression in many quarters.

Similarly, the term "fine screens" has no definite meaning. The author has undertaken to limit the application of the term "fine" to screens which have a clear opening of 0.6 in., this limit being adopted, apparently, in order to include the well-known Hamburg works. In

this distinction, he will not always be followed. There is nothing about the size of the opening which should distinguish one kind of screen from another. The manner of cleaning has more to do with it. As all the screens which have been described in the paper are cleaned with little or no hand labor, the term "self-cleansing screens", or "automatic screens", would be, at least, as appropriate; but why "fine", or "self-cleansing", or "automatic", unless for commercial purposes? In the present state of the art, the simple term "screen" should be sufficiently descriptive. Mr. Soper.

In considering the use of screens, it should be remembered that the screen itself is but a part of the plant which is needed. Screens of coarser size are required in order to protect the finer apparatus against injury and the impairment of its proper function. These must be cleaned, a process which is usually done by hand, and in a simple and crude manner. Grit chambers are needed to perform a like office with respect to the heavy particles, which are recoverable and may do mechanical injury to the screens. Power must be provided to operate the screens, or, at least, their cleansing devices. Storage bins must be supplied for the reception of the screenings, and other provision must be made for their transportation or incineration. Shelter must be given to the plant.

The efficiency should be taken as the action of the whole plant, if the efficiency of screening is to be understood by others than experts.

It is unfortunate that the ways of sampling, testing, and expressing the efficiency are so diverse and unsatisfactory that it is difficult for any one to form a clear conception of what screens are capable of doing. It is a serious question how to test screens and settling basins, and by what criteria to judge them. None of the customary methods seems adequate. The composition of sewage is so various, and its physical, chemical, and biological constituents are so changeable and changing, that it has thus far been impossible to find satisfactory means of determining it. Terms such as "solid matter", "suspended matter", "settling solids", and "suspended organic matter" have very little significance, apart from the behavior of the sewage in some particular respect, in some particular manner, or in some particular case. Laboratory expressions are often misleading. Some of the most dangerous-sounding terms are entirely robbed of their supposed significance when translated into simple language. For example, "nitrogenous organic matter in suspension", may be hair, than which there are few things more incapable of creating foul odors in sewage or of calling for expensive purification works.

The author has performed a difficult and painstaking task in getting together his admirable descriptions of nearly all kinds of screens, whether coarse or fine, and his data are of unusual interest; but the lasting value of his work will not lie in the tables of efficiency.

Mr. Soper. Such data need to be determined by Americans, and for American conditions. It is earnestly to be hoped that those who have charge of sewage experiment stations will see their way clear to install screens of various types and operate them with some not too exceptional sewage for a sufficient length of time for the relative merits of the different forms to become better known. For average sewage, the three forms which have impressed the writer as the most likely to meet practical requirements are those used at Hamburg, Dresden, and Frankfort.

It is acknowledged everywhere that American engineers have done an immense amount of good work in testing out sewage purification processes experimentally, on both a large and small scale, and the reports of some of their investigations are among the most valuable contributions in the literature. Intermittent filters, septic tanks, settling basins, contact beds, and sprinkling filters—all in great variety as to proportions, materials of construction, and rates of flow—have been brought out, have held attention as the leading process of the day, and have been finally placed in the position where they belong. Screening is the last call for notice, and it is to be hoped and expected that it will receive the attention and investigation to which its merits entitle it.

Mr. Allen. KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—The writer wishes to express his gratification at the interest shown in the subject of his paper as indicated by the number and character of the discussions brought out. In particular, these have emphasized: The necessity of standardizing methods of analysis, and the desirability of screening as near the source of pollution as possible.

The views expressed as to the efficiency of fine screening are quite diverse, and are particularly valuable at this time in showing the need of a standardization of procedure in the sampling and testing of sewage and screenings. The Laboratory Section of the American Public Health Association is working along these lines, and within a few years it may be possible to obtain results of screen operation that are safely comparable. The nine efficiencies recorded in Table 12 appear to be discordant. They were obtained from different sewages by different screens operated under different management and the technique in determining the results was probably different in them all. For this reason the writer added the foot-note stating that "More complete data should be secured, in order to furnish a reliable comparison between the efficiencies of different screens." This is in accord with a statement of Mr. Pearse to the same effect.

Since reading the discussions, the writer is more than ever impressed with the futility of placing reliance on efficiencies as ordinarily

* New York City.

published. Mr. Greeley, for instance, states that "The available data indicate that a removal of 15% has seldom been reached in practice." Messrs. Hering, Kershaw, Fuller, and Pearse believe 30% to be, in general, too high a figure; Mr. Stevenson mentions an efficiency of $\frac{200 - 133}{200} = 33\frac{1}{2}\%$, obtained experimentally with a 32-mesh screen at Philadelphia. At Brockton from 50 to 70% is claimed, and Mr. D'Olier states that "Numerous trials of the [Riensch-Wurl] screens, in some fifty plants, show from 32 to 86%, averaging more than 50 per cent." These views are so conflicting that it appears more evident than before that we have much to learn with reference to the true and relative efficiencies of different screens. Mr. Allen.

The subject has been mentioned in so many of the discussions, however, that the writer thinks it desirable to present the following additional information, somewhat in detail, regarding the examples given in Table 12, in order to judge of the relative weight that should be attached to each.

Göttingen, 90 (?) per cent.—This figure was deduced by Mr. Kuichling* from data given by Frühling,† but doubt was expressed by the former as to its reliability, as indicated in Table 12. This result, therefore, may be eliminated from further consideration as probably too high.

Chicago Stock Yards, 63 per cent.—This efficiency was furnished the writer by Mr. C. A. Jennings, in a letter dated January 2d, 1914, as that of his screen operating on Stock Yards effluent. He says: "With an influent running about 700 parts per million of suspended matter, the screen removed about 63% * * * 79% moisture. * * * The above results are the averages of several tests made on the screen." There is no indication that this was a peak load, as assumed by Mr. Pearse. Nevertheless, it was obviously not a normal sewage—a fact which should be borne in mind in considering the subject. Much additional information regarding the operation of this screen may be found in the Report on Industrial Wastes from the Stock Yards and Packingtown in Chicago, by George M. Wisner, M. Am. Soc. C. E., Chief Engineer, and Langdon Pearse, M. Am. Soc. C. E., Division Engineer, dated October 15th, 1914, from which the following notes are taken.

The screen consists of 38 12 by 48-in. panels covered with 40-mesh Monel metal fabric. The net area was 112 sq. ft. With a speed of 125 ft. per min., this passed from 2.1 to 2.7 gal. of sewage containing 340 parts per million of suspended matter per square foot of clean

* "Modern Treatment of Sewage," *Proceedings*, New Jersey Sanitary Assoc., 1908.
† "Entwässerung der Städte," Leipzig, 1910, p. 522.

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screen. The percentage removed, or "computed reduction" was as follows:

October	23d 43%	November	5th 49%
"	24th 32%	"	19th 28%
November	4th 22%		
		Average	33%

This was equivalent to 6 740 lb. of screenings, 83% moisture, or 1 150 lb. of dry matter per million gallons of sewage.

Frankfort, 10 per cent.—This is the figure given by Uhlfelder and Tillmans in "Die Frankfurter Kläranlage." Further information regarding the operation of the Frankfort screens has been received from Stadtbaurat Franze in a letter dated December 30th, 1914. Experiments made in 1908 gave the results shown in Table 26.

TABLE 26.—Removal of Suspended Solids by Grit Chamber and Screens, in Milligrammes per Liter = Parts per Million.

	Grit Chamber.		Screens.		Total.	
	Total.	Organic.	Total.	Organic.	Total.	Organic.
Day sewage.....	65	39	25	24	93	63
Night sewage.....	61	32	5	5	66	37
24-hour sewage.....	64	37	22	19	86	56

"The detritus from the grit chamber is in equal amount organic and mineral, while that from the screens is almost entirely organic. That from the grit chamber is about one-fourth to one-fifth of the entire suspended matter in the raw sewage. The absolute quantities resulting from the foregoing experiments were as follows: [See Table 27.]

TABLE 27.—Material Removed by Grit Chamber and Screens per Day, in Kilogrammes.

	Grit Chamber.		Screens.		Total.	
	Total.	Organic.	Total.	Organic.	Total.	Organic.
Day sewage.....	3 252	1 614	1 317	1 138	4 569	2 752
Night sewage.....	557	292	55	48	612	340
24-hour sewage.....	3 809	1 906	1 372	1 186	5 181	3 092

"As the entire quantity of suspended matter in the sewage at the time of the experiments produced 23 549 kg. of dried substance, we see that the 5 181 kg. recovered by both the grit dredge and screen

indicates that from one-fourth to one-fifth of the entire quantity of suspended matter has been removed by these preliminary operations. Mr. Allen.

"This detritus contains about 80% of moisture, the dried substance being 20 per cent. Considering this quantity of moisture, we have from 5 181 kg. of dried substance, a daily quantity of 25 905 kg. or 25 905 cu. m. (of specific gravity = 1.00) of screenings; or an annual quantity of 365 × 25.905 = 9 528 cu. m.

"Since the installation of the plant in 1905 the following quantities per annum have been removed: [See Table 28.]

TABLE 28.—Material Removed by Grit Chamber and Screens Annually, in Cubic Meters.

Year.	Grit.	Screenings.	Total.
1905	3 604	3 610	7 214
1906	3 208	4 095	7 303
1907	4 365	3 720	8 085
1908	3 582	3 938	7 520
1909	3 859	4 407	8 266
1910	4 320	4 360	8 680
1911	3 745	4 562	8 307
1912	3 608	5 281	8 889
1913	3 062	4 774	7 836
Total	33 353	38 747	72 100

"In 1913 the total volume of sewage was 31 700 000 cu. m. We therefore have the following:

	Liters per cubic meter of sewage.	Cubic yards per million gallons of sewage.
"1. Detritus from Grit Chamber	0.15	0.743
2. Screenings	0.10	0.495
3. Total	0.25	1.238

"The percentage of suspended matter removed by these cleansing operations * * * can be seen in the following table." [See Table 29.]

TABLE 29.—Percentage of Suspended Matter Removed.

	Suspended Matter.				Percentage.	
	Total, in milligrammes per liter = parts per million.		Organic, in milligrammes per liter = parts per million.		Total.	Organic.
Raw sewage		483		287	100	106
a. Grit	65		39			
b. Screenings	28		24			
Total *a* and *b*		93		63	19.3	21.9

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From the foregoing it is seen that the quantity removed per million gallons in 1913 was:

		Percentage of suspended matter. Total.	Organic.
Grit	540 lb.	13.5	13.6
Screenings	234 "	5.8	8.3
Total	774 lb.	19.3	21.9

These percentages are even lower than those given by Uhlfelder and Tillmans, but it must be remembered that the spacing in these screens is 10 mm. or 0.4 in., which is much coarser than any of the others (except Göttingen) for which efficiencies are given.

Table 28 shows that the screenings removed per day in 1913 averaged 13 cu. m. or 17 cu. yd. The figure originally given in Table 12 was evidently incorrect, and has been revised in accordance with this later information. The figure given by Mr. Greeley, 20 to 26 cu. yd., should also be corrected. According to Elsner* the screenings amount to 0.643 cu. yd. per million gallons, or 0.056 cu. yd. per 1 000 inhabitants daily.

With reference to the Frankfort sewage, it is, as mentioned by Mr. Greeley, quite fresh when received at the works. When seen by the writer it was a dark grayish brown, and the screenings consisted largely of feces and paper. The effluent, after passing through sedimentation tanks, flows to the River Main, the width of which is less than 500 ft., where it is diluted by 130 volumes of river water under the most unfavorable conditions of flow.

Strassburg, 10 to 12 per cent.—This percentage is stated as that of the material retained by the screen by Stadtbaurat Strohl in a letter dated May 9th, 1914.

Gleiwitz, 63 per cent.—This (62.9%) is obtained from the data in Table 4, furnished by Magistrat Muler. As stated in the table, the sewage contains "coarse material" and may, therefore, be fresh, explaining the high efficiency, or the latter may be due to the method of sampling.

Mainz, 53.5 per cent.—This is stated by Stadtbauinspektor Knauff† to be the efficiency obtained in a series of tests made by the city authorities, covering a period of several weeks. The sewage is not strong, containing "scarcely 1 ton per million gallons" or about 270 parts per million of suspended matter, but the percentage of removal varied from 46 to 61. These figures, being from an official competitive test, are believed to be entirely reliable, but, for the same reason, they are undoubtedly higher than would have been obtained in ordinary operation.

* "Sewage Sludge," p. 17.

† *Wasser und Gas*, 1911–12, No. 8, p. 183, and *Engineering News*, April 26th, 1913, p. 470.

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Reading, 42 per cent.—This figure is from an estimate by Mr. Kuichling,* showing a removal of 90 parts per million out of 215 parts per million of suspended matter, as follows:

"On a visit to the plant last year, the writer was informed that the weight of the screenings, after partial drying in a centrifugal separator, was 1 500 lb. per million gallons. This would make the condensed mass weigh 80 lb. per cu. ft., of which probably 50% is moisture. If these figures are correct, the screen actually removed 750 lb. of fully dried matter per million gallons of sewage, or 90 parts per million by weight out of a total of 215 parts per million of suspended matter contained in the sewage on the average."

Mr. Fuller's statement that "on a basis of daily averages, this removal rarely if ever exceeded 30 parts per million" is quite at variance with the foregoing, but is probably based on a more recent and intimate knowledge of the operation of this plant, and it is, therefore, probable that the 42% as stated should be modified. The reason for this inconsistency with the data in "Sewage Sludge"† is that they were obtained from different sources.

Brockton, 71.3 per cent.—Regarding this screen, Mr. Charles R. Felton, then City Engineer, wrote on November 15th, 1912:

"The screen has been in continuous operation now more than a year, with practically no stops—not over an aggregate of two or three days in that time except the hour or two that we stop every day. The screening [mesh?] has not been replaced in that time. The time for cleaning the mesh occupies perhaps 15 or 20 minutes every morning. The detritus removed, in our case, amounts to from five to six thousand pounds per day after having been passed through a centrifugal dryer, from about 2 000 000 gal. of sewage. Our sewage, as you know, is very strong, on account of the water consumption, namely, about 40 gal. per capita.

"This five or six thousand pounds of screenings is equal to about four cu. yd. in that form. In our particular case it is necessary to raise the sewage into the screen, which is done by a 12½ h.p. oil engine which also revolves the screen. It is necessary to have one man constantly at the plant during the 24 hours, so that in working 8-hour shifts three men are necessary in the entire day."

The figures on which the efficiency were based were referred to X. H. Goodnough, M. Am. Soc. C. E., Chief Engineer of the Massachusetts State Board of Health, who states, April 9th, 1914:

"* * * the results as stated in our report for 1912, and as copied in your letter, are correct.

"There is danger of great error in conclusion about this screen unless the proper system of sampling is carefully carried out. If the

* Given in "Notes on Sewage Disposal," Rochester, March, 1910, p. 15.

† Pointed out by Mr. Pearse. Using 128 parts per million of suspended solids, as given in "Sewage Sludge," would give $\frac{26}{128 + 26} = 17$ per cent.

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samples are taken mostly in the day time they will represent a night flow of sewage and consequently a much weaker sewage than if the samples are taken throughout the 24 hours. We have some reason for thinking that the sampling was done with less care in 1913 than in the previous year."

Mr. Goodnough writes again, February 4th, 1915:

"The figures given in our report* showing screening results at Brockton in 1912 were made up by taking samples generally once each month, though there were, I believe, a number of extra samples taken in 1912. These samples were taken by employees of the Brockton Sewer Department at the pumping station, and each sample is made up of twenty to twenty-four equal portions collected approximately hourly throughout the twenty-four hours.

"The sewage strikes the screen with a good deal of force and, in consequence, the mesh after use for a time became broken in places, and this no doubt accounts for the poorer results subsequently obtained. Furthermore, the screen at Brockton is not so arranged that the entire screen is used, most of the screening being done by the first section.

"There was a change in management in 1913 which may have had something to do with the falling off in efficiency. There was again a change of management in 1914, and I send herewith the figures for 1913 and 1914, obtained in the same way as in 1913 [all in parts per million].

	Suspended Solids.			Albuminoid Ammonia.			Oxygen Consumed.		
	Raw.	Screened.	Removed.	Raw.	Screened.	Removed.	Raw.	Screened.	Removed.
1913	500.5	223.0	55%	16.4	12.4	24%	137.1	122.8	10%
1914	642.4	250.7	61%	21.6	12.5	42%	160.6	125.0	22%

"The authorities at Brockton took samples for awhile to check our own, but finding the results much the same, concluded that they would take our results and not take the trouble to carry on determinations to check them. So far as they did make observations, they appear to have checked ours very closely * * *.

"It is important to note in connection with the efficiency of screening in Brockton that Brockton sewage is one of the strongest in the State. * * * the amount of total solids * * * is exceeded only at Hudson and Norwood and * * * it exceeds somewhat the amount of solids in the sewage at Worcester. The amount of albuminoid ammonia also is much higher than the average. * * * In considering the results for 1914, it is necessary to take into account the fact that the year 1914 was a very dry one after May 13th, and you will note that the amount of albuminoid ammonia in the raw sewage examined in that year was higher than in any other, except the sewage of Hudson, while the total solids were higher than in any other sewage examined in that year.

"We have not had experience enough with screening to know its exact effect with different sewages. It is possible that the percentage

* Massachusetts State Board of Health, 1912.

of organic matter removed is higher with a very strong sewage like that at Brockton than would be the case with a much weaker sewage such as is found in some of the other cities of the State." Mr. Allen.

Further information regarding the Brockton screen is contained in a letter from Mr. B. R. Chapman, City Engineer, dated February 6th, 1914. He says:

"* * * our Weand screen has been in continual operation, nearly every day (since 1912), which gives a very limited time for proper repairs; the frame of the screen is now in bad shape and some of the sewage does not pass through it at all, so that the reduction of suspended matter may be estimated at 50%.

* * * * *

"The cost of operation for 1914 was approximately $8 500, which includes attendance, oil for oil engine, water for cleaning screen, repairs, miscellaneous supplies, and also the carting of screenings to boilers to be burned.

"The capacity of the screen is about 3 000 000 gallons daily. This screen, I believe, has several mechanical defects and might easily be improved upon * * *."

A Weand screen operating on Center Avenue sewage (Packingtown), Chicago, was 4 ft. 8 in. long and 2 ft. 4 in. in diameter, with 29.3 sq. ft. of net area, covered with 30-mesh fabric. This was operated at a speed of 7 rev. per min., and received from 117 000 to 235 000 gal. per day, or from 4 000 to 8 000 gal. per sq. ft. of exposed screen. This is equivalent to about 0.4 to 0.8 gal. per sq. ft. of clean screen exposed. In twenty-seven experiments the removal of suspended solids amounted to 32% "computed" or 17% "actual" on day sewage. Continuing operation into the evening reduced the former to 12 per cent. There were removed by this screen 500 lb. of dry matter or 3 cu. yd. of sludge, 90% water, per million gallons.*

Dresden, 33.6 per cent.—This efficiency is by volume; the efficiency by weight is 30.95 per cent. Full details are given in Table 10, furnished by Baurat Fleck. This is the average result of a complete year of operation, samples being taken every fourth day at 4-hour intervals above and below the screen. Considering the care with which this plant is managed and the length of the test, the writer attaches more weight to the results obtained here than to those obtained elsewhere. The figures are corroborated by the results of the test of a Riensch-Wurl screen at Mainz (see page 1892†) where the efficiency, with a sewage of moderate strength, varied from 36 to 42 per cent. As to the distance from the source to the outlet, the situation in Dresden is probably not very different from that in the average American city. This may be seen by reference to Fig. 43. From Table 15, by Mr. Kershaw, there result the following seasonal efficiencies at this plant: spring, 18.7%; summer, 17.3%; autumn, 27.6%; winter,

* Report on Industrial Wastes from Stock Yards and Packingtown, 1914.

† *Proceedings*, Am. Soc. C. E., for August, 1914.

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48.3%; showing, in a rough way, the effect of temperature on the condition of the sewage.

The sewage when noted was a dark greenish-gray color. The screenings removed by the coarse screen were mostly composed of paper, rags, and sticks. Mr. Greeley's figures, 13 to 16 cu. yd. of wet screenings per day, do not agree with the volume reported by Stadtbaurat Fleck in a letter of April 9th, 1914, which was 19.7 cu. m. = 25.6 cu. yd.

Referring to Table 24, the quantity, 0.97, in Column 3, is in tons, and, adopting Mr. Greeley's assumption of 60 lb. per cu. ft., should be 1.20 cu. yd. This gives 12.4% removal instead of 10.1 as stated, if the parts per million of suspended matter are taken at 300. The authority for these assumed figures for weight per cubic foot and parts per million is not stated.

The writer believes that he is justified in giving preference to the official results of the year's operation stated in Table 10, as follows: 33.64% efficiency on the basis of volume or 30.95% on the basis of weight, rather than to a computation in which two of the factors appear to be assumptions.

Leaving the 90% of Göttingen out as unreliable, the 10% of Frankfort as inapplicable, on account of the coarseness of the screen, and, omitting the 63% of the Chicago Stock Yards, because of the abnormal character of the sewage, there remain six examples, in five of which, the efficiency as stated exceeds 30%, reaching 71.3% in the case of Brockton.

After making due allowance for errors of sampling and analysis, and considering that the efficiencies given are not selected, but a complete list of all the writer could find on screens of these types, he cannot agree with Mr. Greeley that "the available data indicate that a removal of 15% has seldom been reached in practice." It is certainly reached in Europe (for example, 66% at Wiesbaden*) and at Reading and Brockton, although, so far, experience in America is very limited.

The writer, therefore, believes that his assumption of a removal of 30% was not unreasonable for screens of the types described, having openings not more than 0.10 in. in size, under efficient operation. He believes, however, the 42% credited to the Weand screen at Reading is probably too high, and that data concerning efficiencies in general are far too meager to make any prediction with confidence. Higher values would naturally be looked for in Europe, with the stronger sewages prevalent there, but, as to length of carriage, conditions are probably not very different between European and American cities. Figs. 40 to 43† show the main lines of collection in London, Hamburg,

* Weyl's "Handbook of Hygiene," Vol. II, p. 151.

† From Final Report, Metropolitan Sewerage Commission of New York, 1914.

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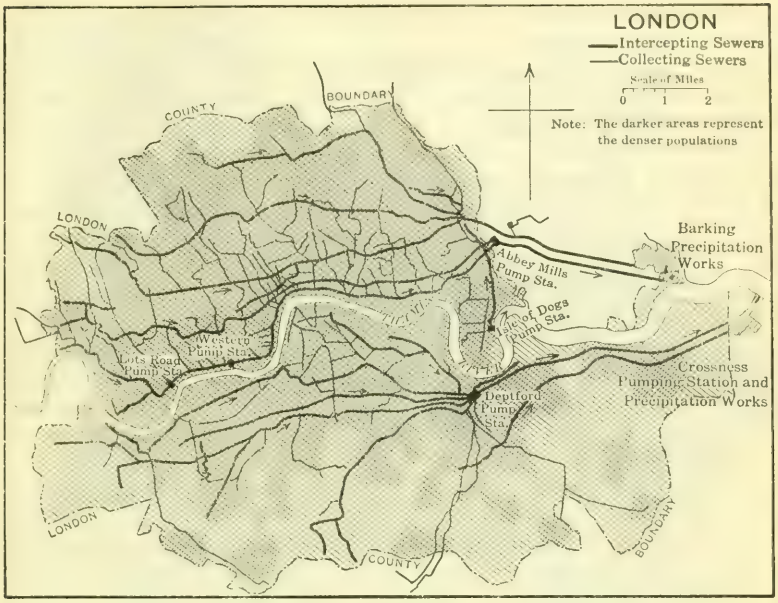

FIG. 40.

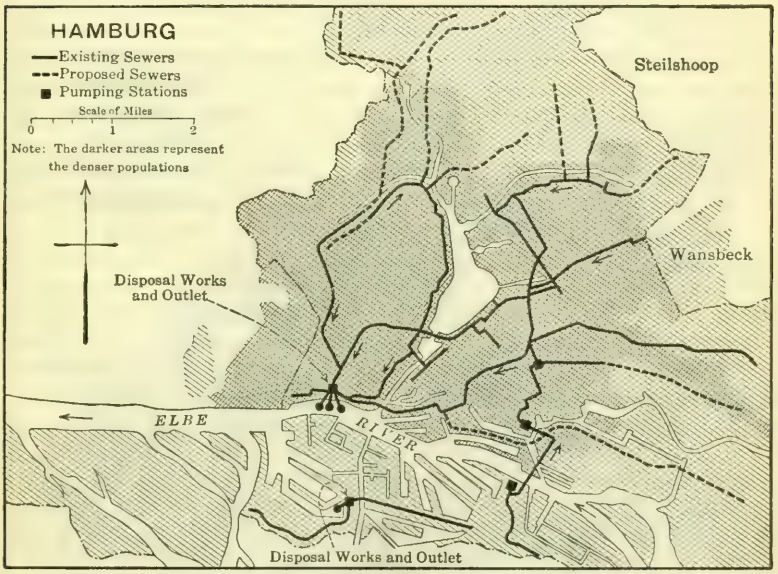

FIG. 41.

Mr. Allen.

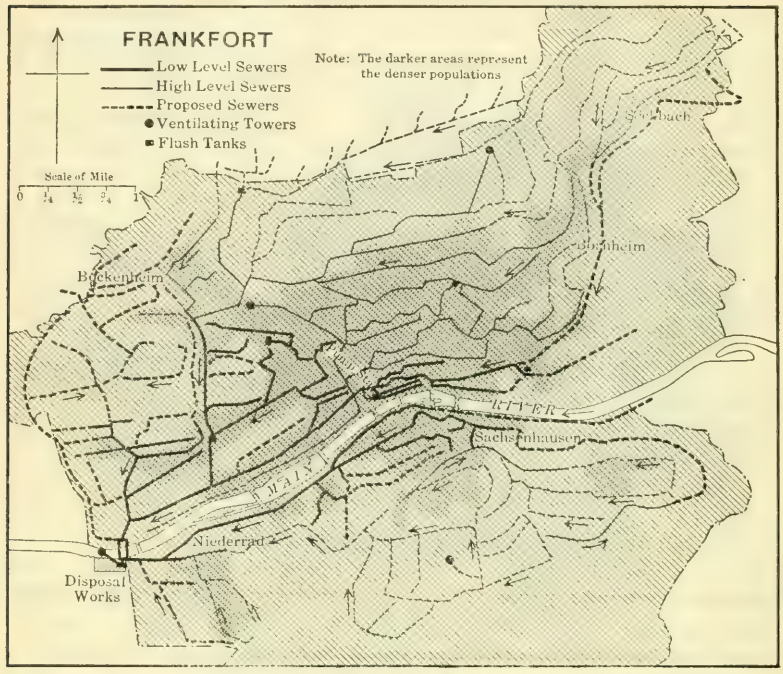

FIG. 42.

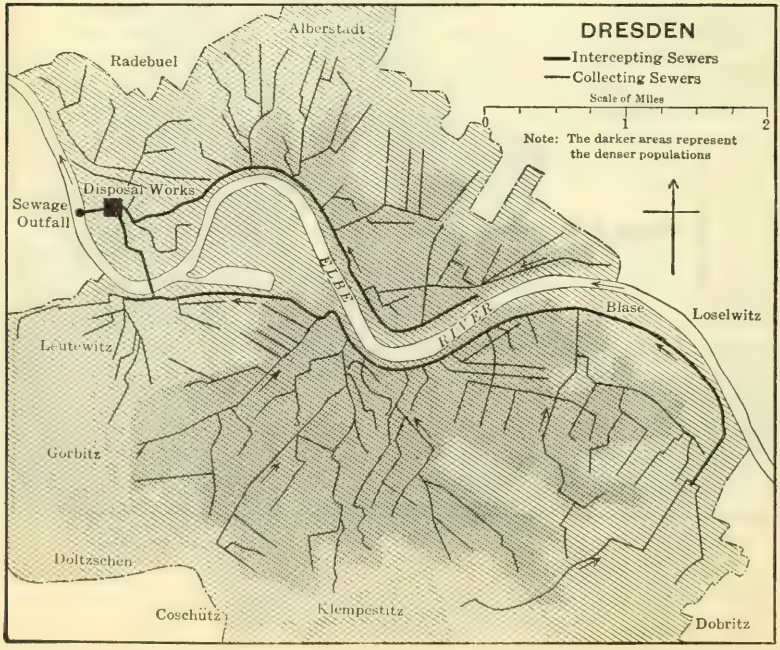

FIG. 43.

Frankfort, and Dresden, from which can be obtained a good idea of the distance traveled. Mr. Allen.

This point, emphasized by Mr. Hering and mentioned also by Messrs. Fuller, Franze, Schaefer, Soper, and Kershaw, is believed to be very important. It is evident to any one who has observed the effluent from long outfalls or those from septic tanks. At Paris the 170 000 000 gal. per day received at Clichy is quite free from feces, the principal materials removed there being some 2 cu. m. (2.6 cu. yd.) of corks per day, which are recovered and manufactured into steam pipe covering, 150 cu. m. (195 cu. yd.) of grit, and 30 cu. m. (39 cu. yd.) of screenings. The latter, when seen in May, 1913, were composed almost entirely of vegetable matter: leaves, straw, orange peels, cabbage leaves, and sticks, with more or less paper, the feces having been thoroughly comminuted. Probably the character of the sewage received here is also influenced by the local screening which has been so interestingly described by Mr. Granbery.

At the Boston and London outfalls, also, there is comparatively little visible evidence of fecal matter. As pointed out by Messrs. Potter and Hammond, as a sewage becomes septic, the relative advantages of screening are lessened. In some cases, as with the Elberfeld-Barmen plant, the quantity of screenings removed, even after a long travel, may be large, due perhaps to the inhibition of decomposition by trade wastes.

Messrs. Kershaw, D'Olier, Potter, and Jennings regard the percentage of removal as of doubtful importance, stress being laid on the condition of the effluent as the real criterion of screening, regardless of the proportion of suspended matter removed. This test would appear to have decided advantages, for the efficiency is known to fluctuate with the strength of the sewage to a much greater extent than the suspended matter in the effluent, which, after all, is the important consideration. In comparing the results obtained by different plants, the strength of the sewage should, of course, be kept in mind, as mentioned by Messrs. Pearse and J. H. Gregory with reference to European *versus* American sewages, but the suspended matter in the effluent will not be proportional to the strength of the sewage. In other words, though a higher efficiency would be expected from an average European sewage, the suspended matter in the effluent would probably vary to a much less extent from that from an average American sewage, provided the screens and methods of operation were similar.

Some interesting experiments were made with different screens on the same sewage in Chicago by Messrs. Wisner and Pearse for the Sanitary District. The results were, in brief, as shown in Table 30.

The Philadelphia experiments described by Mr. Stevenson furnish valuable information of practical application, confirming what has

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been said regarding the favorable results of screening in preventing the clogging of nozzles, also on the favorable character of the subsequently settled sludge for rapid drying; Messrs. Hammond and J. H. Gregory mention its use in preventing scum formation; Messrs. Whittemore and J. H. Gregory on the consequent reduction of sludge storage; Mr. Whittemore of the reduction of the area of sludge-drying beds; and Messrs. Whipple and Stevenson point out the increased efficiency obtained in the disinfection of effluents by the use of fine screens. These are all important considerations favoring screening, based on actual experience.

TABLE 30.

Size of slot, in inches	1.00 by 0.0625					0.50 by 0.032			0.050 by 0.025	
Average reduction of suspended matter	14%					20%			25%	
Time required to reach loss of head of 1.5 ft. at rate of 11 100 gals. per sq. ft. daily	28 min. 45 sec.					9 min. 8 sec.			7 min. 14 sec.	
Meshes per linear inch	6	10	16	20	24	30	40	60	80	100
Average reduction of suspended matter	13%	10%	17%	16%	21%	20%	26%	23%	33%	29%

The method of estimating efficiencies from the dry material in the screenings and the suspended matter in the effluent suggested by Mr. Kershaw and used by Messrs. Pearse and Greeley, appears to the writer to have distinct advantages in avoiding erratic results due to large masses of suspended matter included in, or omitted from, the sample. The writer is not an analyst, and does not feel competent to express a more definite opinion on this procedure, but if it should prove that results heretofore reported are unreliable for this cause he is most anxious to be made aware of it. The drop in efficiency at Dresden from 66.4% in 1912-13 to 30.95 in 1913-14, as given in Tables 9 and 10, was attributed mainly to the mode of sampling.

Several of those discussing the paper lay stress on the desirability of arriving at some generally acceptable method of sampling, and of estimating screen efficiencies, mentioning at the same time the great difficulty in doing this on account of the number and uncertainty of the factors involved.

Messrs. Franze, Schaefer, and Kershaw point out that screening has little effect in reducing putrescibility and Messrs. Soper and Hering that its effect on the reduction of turbidity is negligible. These statements are very true, and should be kept in mind in selecting a method of treatment. In fact, as mentioned by the writer and by Mr. Potter, the colloidal matter may be increased by screening. On the other hand, screening removes the more apparent sources of pollution and, as mentioned by Messrs. Hering and Hammond, permits of a lower ratio of dilution of the effluent without offense.

Mr. Potter is of the opinion that the increase of solids in solution, as computed for Gleiwitz, is in error. If the dried residue after filtration, as stated in Table 4 (which represents the solids in solution), is increased by screening from 1 237.2 to 1 260.4, this 1.9% of suspended solids is correct and the writer is unable to see how any qualifying effect due to stable solids in solution is admissible. The particular figure in any one case is of little importance, emphasis being placed on the fact that under certain conditions a portion of the suspended solids may become colloidal or pass into solution in the process of screening. In fact, the writer agrees with Mr. Whittemore, that precise determinations are so uncertain that it is an unnecessary refinement to express percentage efficiencies in less than whole numbers. Mr. Allen.

Mr. Pearse says:

"Comparative results on the same sewage do not indicate that screening, even with a 30-mesh screen, is the equivalent of thorough sedimentation, nor within 50 per cent."

On close inspection, the writer fails to see where any such assumption was made. On page 1842* he expresses his belief that the time has come "when screening may fairly be compared with tank treatment", but, that he did not consider them equivalent is evident from his statement on page 1905*:

"* * * the best fine-screening will compare well with such tank treatment as may usually be looked for: That is, from 30 to 50% of the suspended solids may be removed by fine screen, as compared with 50 to 65% by sedimentation."

Although the writer believes that the former figures are higher than it would be wise to assume in planning new work, they still leave a fair margin between the best screening and sedimentation, as ordinarily carried out.

Assuming 1.9 cu. yd. of screenings containing 375 lb. of dry substance per million gallons of sewage, as deduced from Table 12, and 1 527.5 lb. of dry matter per million gallons of sewage, as found at Providence with plain sedimentation, Mr. Kuichling has estimated a probable removal by fine screens of 46.65% of the removal by settling in tanks.

Mr. Pearse infers that, in comparing screening with tank treatment, the writer has considered openings as large as 0.6 in., whereas these are specifically stated as "not more than 0.10 in. (2½ mm.) in size".

With regard to the whole question of screen efficiency and the relative value of screening *versus* settling, the writer desires to say that he has no axe to grind, in either connection, and can express his

* *Proceedings*, Am. Soc. C. E., for August, 1914.

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attitude no better than by repeating the following statements from the paper:

"* * * it is the writer's opinion that fine-screening will be adopted in the future by many towns situated on bodies of water which are capable of assimilating the effluent; possibly, also, as a preliminary process to tank treatment, filtration, and disinfection. On the other hand, the relatively high cost of attendance and the probable lack of a market for the screenings, when compared with conditions abroad, will probably serve to prevent the marked increase in their use in the United States which has been experienced in Germany."

The discussion by Mr. Soper furnishes an excellent summary of the relative advantages of each method of treatment.

The writer is sorry that more data regarding screen efficiencies have not been forthcoming, and realizes more than ever that additional information must be had before any figure can be viewed as other than tentative.

There is one more point: In any consideration of fine screening it would seem proper to include the auxiliary process of grit removal, which may or may not be required. In other words, in a comparison of plants, these should be considered complete, with all the accompanying features depending thereon, not only grit removal, but power, disposal of screenings, and attendance. This view is also expressed by Mr. Soper.

Mr. Kershaw mentions the deposit of grit as possible where the current above the screen is retarded by the screen itself. This is important. The velocity above the screen should be sufficient and that through the screen not so great as to force the larger solids through. With the U-shaped channels of the drum and Riensch-Wurl screens, and the method of scooping up from the bottom with the shovel-vane screen the likelihood of deposits near the screen is reduced. As a general proposition fine screens should have grit chambers placed above them for their protection.

Mr. Kuichling's suggestion that screens be cleaned by suction is one which certainly deserves serious consideration. If worked out so as to be a mechanical and economic success it would appear to be ideal. To the writer, however, there seems to be little objection to the methods used at Hamburg, Dresden, and Frankfort, so far as odors, spray, or other sanitary considerations are concerned.

The question of head consumed has been mentioned by Messrs. Greeley, Kuichling, Pitkethly, and Riedel. Answering Mr. Kuichling's question as to the coefficient of friction, 0.4, the writer quotes as follows from a letter of March 2d, 1914, from Mr. H. H. Stevens, of the Sanitation Corporation, Philadelphia.

"Mr. Wurl's method of figuring the loss of head is to consider about one-fourth of the submerged area as being effective opening

and to assume one-half to two-thirds of this effective opening being covered by screenings in order to arrive at the true effective opening for passing the effluent through the screen, and then computes the loss of head on the basis of the formula, Mr. Allen.

$$H = \left(\frac{Q}{C \sqrt{2\,G} \times A} \right)^2,$$

using the coefficient, C, at a value of 0.4, which he claims has been proven by test."

He adds that, on account of the weaker American sewage, the Sanitation Corporation figures that about one-fourth of the effective opening will be covered by screenings. A coefficient of, say, 0.6, would undoubtedly correspond more nearly with what meager data on small ajutages are available, but the use of a smaller coefficient is no doubt preferable in providing a greater factor of safety. The writer, however, is not responsible for the particular value mentioned. He regrets that he cannot give other figures for drop in head besides those already mentioned, except in the case of the Drum screen used in the Mainz experiments mentioned on page 1875* in which this varied from 1.4 to 7.3 in.

Mr. Hammond and Mr. C. E. Gregory speak of the condition of the Thames, Elbe, Rhine, and other European rivers which receive sewage effluents from large populations without resort to the oxidizing processes often supposed to be necessary. At London and Glasgow the effluents from precipitation plants appear somewhat turbid if more than 1 or 2 ft. in depth, generally of a reddish-brown color, but without visible solids, grease, or odor. At Crossness (London), partly due to the brownish-green color of the river, the effluent could be followed for several hundred feet from the outlet, but no other sign of pollution was noted, such as sleek or floating solids.

At Frankfort, after fine screening and plain sedimentation, the effluent appeared to be free from solids, but was somewhat cloudy when of much depth. The raw sewage was dark grayish-brown.

The writer was unable to discover the effluent when passing near the three submerged outlets at Hamburg. Mr. Kuichling, has stated:†

"The result has been entirely satisfactory, and it is now reported to be almost impossible to detect any evidence of sewage pollution in traversing the river with a vessel. The suspended matters in the sewage that pass through the screens seem to remain near the bottom, and to be finally carried out to sea by the current."

In spite of the coarseness of the screens used at Hamburg, the absence of floating matter is also confirmed by Mr. Hammond. It should be said, however, that the river itself is somewhat brownish

* *Proceedings*, Am. Soc. C. E., for August, 1914.

† "Notes on Sewage Disposal," Rochester, 1912, p. 19.

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and turbid. If clean and saline, the effluent would be much more apparent. According to H. N. Ogden, M. Am. Soc. C. E., the manager of the plant stated that "very seldom and only in the winter could anything in the nature of a grease film upon the river surface near the outlet be detected".*

Experiments made by the Metropolitan Sewerage Commission of New York, by discharging sewage, or fresh water colored by a strong dye, at different depths below the surface illustrated in a very convincing manner the influence of salinity on the appearance of sewage effluent at the surface. With fresh river water, as at Hamburg and Washington, there is much less tendency to rise, and diffusion is more prompt; but with a saline water such as that of Boston (Nut Island outlet) or New York Harbors, the sewage rises promptly to the surface, where it spreads out in a thin sheet and is made more apparent by the saponification of the grease. This difference between river and ocean water with respect to sewage effluents should always be kept in mind.

Mr. Kershaw's inquiry regarding stream flow and dilution is answered in part by the following notes.

At Barking and Crossness some 400 000 000 gal. of London's sewage per day, from a population of probably 6 300 000, is discharged into the Thames after being treated by chemical precipitation. The river here has a tidal range of some 20 ft. and a net flow toward the sea of about 190 000 000 cu. ft. per day.† This furnishes a dilution, in addition to that derived from the oscillation of the tide, of about 5.1 volumes, or 3.5 cu. ft. per sec. per 1 000 persons. The average salinity, as computed by the writer, is about 200 grains per Imperial gallon, which is equivalent to a mixture of 5 parts of river to 1 part of sea water.‡ Although there is a satisfactory absence of floating solids in the water, apprehension has been felt concerning the depletion of oxygen in the Thames near the outfall where, at times in summer, it is nearly complete.

At Frankfort, the River Main receives the tank effluent from a population of 420 000. This effluent contains only 20% of the suspended matter in the raw sewage, half of which is non-sedimentable. The latter is variously stated as 411 and 536 mg. per liter = parts per million.§ The flow of the Main and the corresponding volumes of dilution are as follows:

During	low	water	=	6 400 000	cu. m. per day	=	128 volumes
"	mean	"	=	15 000 000	" "	=	302 "
"	high	"	=	83 800 000	" "	=	1 676 "

* *Engineering News*, October 13th, 1910, p. 386.

† "Movement of the Water in a Tidal River," W. Cawthorne Unwin, 1883.

‡ Report on Thames Conservancy by Medical Officer of Health, 1907.

§ Uhlfelder and Tillmans, "Wasser und Abwasser," Vol. I, No. 7.

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At Dresden the average flow of the Elbe is somewhat less than 79 cu. m. or 2 790 cu. ft. per sec. The water contains 6.2 tons of solids per 1 000 000 cu. ft., and the sewage 37.4 tons. After receiving the effluent the river water contains 7.8 tons per 1 000 000 cu. ft. It is expected, when the population reaches 800 000, that to 293 cu. ft. of solids brought down by the stream, 86 cu. ft. will be added by the sewage effluent.

At Hamburg, 53 000 000 gal. of screened effluent per day from about 1 000 000 inhabitants are discharged in the bottom of the Elbe, the low-water discharge of which is here 5 200 cu. ft. per sec. or 5.2 cu. ft. per sec. per 1 000 population, and the mean discharge 23 000 cu. ft. per sec. or 23 cu. ft. per sec. per 1 000 population. Therefore, there is a dilution of 63 volumes at low water and of 280 volumes at times of mean flow.

The automatic control of speed which Mr. Kuichling has mentioned as so desirable with a variable flow, is accomplished with the Carshalton screen, the motion of which depends on the undershot wheel actuated by the flow of sewage. In the other types of screen this may be accomplished by auxiliary mechanism if desired, although there is a certain compensation in most screens by the increased area of screen surface submerged with an increasing flow.

The writer admits that in the paper the subject of the disposal of screenings was not given the consideration it deserved, and is glad to see that it is taken up by several of those who contributed to the discussion.

The quantities obtained at Hamburg, Frankfort, and Dresden, as given by Mr. Greeley, differ materially from those in the paper, namely:

	Hamburg.	Frankfort.	Dresden.
Allen.....Per day	18 cu. yd.	50 tons	25.6 cu. yd.
Greeley....Per day	26 cu. yd.	20-26 cu. yd.	13-16 cu. yd.

These differences may be due to errors in information furnished, or to altered conditions, but with regard to the writer's figures he would say that that for Hamburg was obtained from Baudirektor Sperber in a letter of April 2d, 1914. That for Frankfort was obtained verbally at the works, was possibly misinterpreted, and is believed to be incorrect. A preferable figure is 17 cu. yd., being the average daily output in 1913, and obtained from Stadtbaurat Franze under date of December 30th, 1914. The average volume obtained at Dresden was given for 1913 by Stadtbaurat Fleck in a letter of April 9th, 1914.

The possibility of a nuisance from odors with the storage of screenings mentioned by the writer, has been spoken of by Messrs D'Olier, Potter, Greeley, J. H. Gregory, and Soper. In this respect, well-digested sludge has a decided advantage, as mentioned by Messrs. Hammond and Potter, but the satisfactory disposal of screenings is believed by the writer to be a simpler matter than that of ordinary settled sludge or that

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from chemical precipitation. It is essential that means be provided for prompt removal and disposal in conjunction with screening plants, but there should be no serious difficulty in doing this, by dewatering and burning, or otherwise. The odor of screenings stored for several days resembles that of a pig-stye, and would, under certain conditions, be objectionable at a distance of several hundred feet. At Cologne, the screenings are stored in bins about 100 ft. long, 11 ft. wide, and 4½ ft. deep. Two or three of these were filled when visited in June, 1913. The daily output is about 21 cu. yd. per day. Flies were abundant, and the odor was very unpleasant. Under certain conditions, this was said to be noticeable at a distance of nearly 1¼ miles; at Dresden, ¼ mile. The screenings are valued as a fertilizer, but their storage is a serious nuisance. The cost of disposal, naturally, should be included with the other costs of operation, in comparing different schemes.

Mr. Kershaw mentions the availability of screenings as a manure. This may be practicable in some instances, as the screenings undoubtedly have a fertilizing value, but there should be some assurance that they will be removed regularly and not stored where likely to give offense. An analysis of the Hamburg screenings is given in Table 1. The following is given by Tillmans* for Frankfort:

	Grit.	Screenings.
Moisture	60-70%	80%
Of the dried material:		
Nitrogen, N	0.12%	0.82%
Phosphoric acid, P_2O_5	1.01%	1.57%
Potash, K_2O	0.12%	0.40%

At Wiesbaden the screenings are stored under a shed and covered with sweepings, where they remain until taken by neighboring farmers. By some such way or by composting the danger of nuisance may be reduced.

In Germany it may be said that the utilization of screenings as a fertilizer is the ruling custom, but where the percentage of fats is as high as at Dresden, where it is said to be 18%, there appears to be a promise of profit in its recovery. Here the screenings are first drained, shrinking to one-third their original volumes, and have recently been heated in a rotary dryer, treated chemically to remove the grease, and the products sold at a net profit of $15 000 per annum.

The cost of the Frankfort screening plant, recently received from Stadtbaurat Franze, is as follows:

Grit elevator and motor	6 600 Marks	= $1 570.80
Three automatic screens, with motor and appurtenances	21 600 Marks	= 5 140.80
Apparatus for removal	4 800 Marks	= 1 142.40
Total	33 000 Marks	= $7 854.00

* Water Supply and Sewage Disposal.

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Disposal by incineration, with refuse where practicable, as at Frankfort, and mentioned by Messrs. Franze and Schaefer, has the advantage of being inoffensive and practically without cost.

Costs of installation and operation of German plants, as mentioned by Messrs. Cohen and Potter, are not directly applicable to American conditions, but are valuable in matters of comparison between each other and, after allowing for differences of ruling wages, etc., as a guide for projected work elsewhere.

In reply to Mr. Kershaw, the figures for cost of operation that have been given do not include charges for interest or amortization. In a comparison of different schemes these, of course, should be included.

Regarding the very high cost given for the Dresden plant ($2 150 000), it should be explained that this covers the cost of grit chamber, elevators, administration buildings, workshop, heating, power and electric lighting plants, and all appurtenances besides that of the screens alone.*

Mr. Pearse asks why 0.6 in. was taken as the dividing line between coarse and fine screens. The precise limit is quite arbitrary. The writer agrees that, say, 0.2 or 0.15 in. would for many reasons be better. In the paper 0.6 in. was adopted so that the Hamburg screens might be included in the comparison, both on account of the importance of these screens and of the band type of screen in general, others of which are of much finer opening.

Messrs. Hering and Cohen mention the use of screens before centrifugal pumps. Perhaps a more positive advantage would follow in their use before triplex pumps, which, as a rule, operate at higher efficiencies and are more likely than centrifugal pumps to be injured by coarse solids.

Regarding the applicability of fine screens to New York City, referred to by Mr. D'Olier, the conditions at different outlets and suggested points of disposal vary very greatly. For some of these a high degree of treatment will probably be required eventually, for others tank treatment will be sufficient, and it is probable that there are yet others where fine-screening will prove most advantageous. In parts of Manhattan the sewage is (for American sewage) very strong, besides being very fresh, both of which considerations favor this method of treatment.

As to the type of screen, the writer feels that, from lack of familiarity, the Windschild and Geiger screens have not received the recognition in the discussion that they deserve. Probably no one type can be pronounced as best; the selection should depend on local conditions and prices.

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* *Municipal Journal*, December 28th, 1911.

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In closing, the writer thinks it appropriate to mention the work done by the late Emil Kuichling, M. Am. Soc. C. E., in collecting European data on fine-screening. This was done with his customary thoroughness, and presented in a practicable form for the use of American engineers at a time when fine-screening was scarcely known here. With his keen insight, he appreciated both the possibilities and the difficulties to be overcome in the use of fine screens, and was one of the first to advocate their adoption in America. Much of the writer's interest and information regarding them is due to Mr. Kuichling, who was always ready and willing to spare his valuable time to discuss the subject, and it creates mingled feelings of regret and appreciation to note that his discussion of this paper was the last work done by him.

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PAPERS AND DISCUSSIONS

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT

Discussion.*

By Messrs. M. M. O'Shaughnessy, J. D. Galloway, F. G. Baum, and E. G. Hopson.

M. M. O'Shaughnessy,† M. Am. Soc. C. E. (by letter).—Mr. Grunsky, in his discussion, states correctly that in Mr. Cory's paper there is a marked departure from the ordinary treatment by an engineer of an engineering problem. At and previous to the time the paper was prepared its author had never been officially employed on an investigation of the subject therein discussed. It is therefore natural that any engineering data presented should be merely a résumé of the studies of other investigators, and since no new light is thrown on the subject, the matter therein is of little importance to the profession at large. Mr. Grunsky, however, as noted in his discussion, held the position of City Engineer of San Francisco from 1900 to 1904. As the former occupancy of that office lends weight to his statements on the water question, it is necessary that some of the erroneous impressions conveyed by his discussion be corrected. Mr. O'Shaughnessy.

Mr. Grunsky calls attention to the fact that the Hetch Hetchy project under the Raker Bill is not his Hetch Hetchy project. Despite Mr. Grunsky's best efforts, his application, filed in 1901 with the Secretary of the Interior, for a permit to impound water at Hetch Hetchy and Lake Eleanor and conduct 60 000 000 gal. daily to the City was denied. In 1905 the application was renewed, but premission was again refused.

In 1908 Secretary Garfield issued a revokable permit allowing the construction of the Eleanor Dam as a first unit of construction,

* Discussion of the paper by H. T. Cory, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† San Francisco, Cal. Previous discussion on this paper by Mr. O'Shaughnessy was printed in *Proceedings* for January, 1915.

Mr. O'Shaughnessy.

but requiring the full development of the Lake Eleanor water-shed before any work could be done in the Hetch Hetchy Valley. Only 79 sq. miles of water-shed are tributary to the Eleanor Dam site, on which a dam, 1 700 ft. long on the crest, would have to be constructed to impound a run-off of only 20 billion gallons in ordinary seasons, and in years of minimum precipitation 8.7 billion gallons. In contrast, the area tributary to the Hetch Hetchy dam site is 459 sq. miles, or 5½ times that of Lake Eleanor, and a dam 300 ft. high and 700 ft. long will impound 112 billion gallons. The Eleanor Dam site is much more inaccessible than the Hetch Hetchy, and unit prices for construction materials delivered would be correspondingly greater. Despite the fact that the Garfield permit would require a vastly greater initial expenditure to develop far less water, and after the City had voted $45 000 000 for constructing the Hetch Hetchy-Lake Eleanor system, in February, 1910, Secretary Ballinger called on San Francisco to show cause why the Hetch Hetchy Valley and reservoir site should not be permanently denied San Francisco and eliminated from the above permit. After fighting for nine years, the City had nothing tangible to show for the money it had expended. Then, to quote Mr. Grunsky:

"San Francisco was thus put on the defensive and, due to lack of confidence of various of her official departments in each other, placed this case in the hands of an expert called in from the East."

This expert, of whose employment Mr. Grunsky declares with some bitterness "the natural result was that thousands of dollars were needlessly expended in the accumulation of a mass of statistical information", was John R. Freeman, M. Am. Soc. C. E., whose masterly presentation of accurate statistical information on this project to the Board of Army Engineers was one of the most potent factors in securing for San Francisco the Hetch Hetchy grant. The subsequent report rendered to the Secretary of the Interior by the Board of Army Engineers substantiated practically every claim made by Mr. Freeman. Mr. Freeman's report, augmented by the oral testimony of the City Engineer of San Francisco, Percy V. Long, City Attorney, and Colonels Biddle, Cosby, and Taylor, who constituted the Advisory Board of Army Engineers, resulted in the enactment by Congress of the Raker Law.

Mr. Grunsky speaks of extraordinary expenditures and lack of study. He neglects to mention that if there were any extraordinary expenditures he was a beneficiary thereof as he received compensation for studies of the Mokelumne River, the McCloud project, and for other services. Possibly Mr. Grunsky's professional pride is hurt because his report on the Mokelumne River was not presented by the City when the case was pending before the Secretary of the

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Interior. The report was so hastily and disjointedly prepared, and contained such insufficient data that Mr. Freeman and the Army Engineers gave it no consideration. This, together with the fact that Mr. Grunsky's original plans were radically changed, undoubtedly inspires much of his criticism.

Under the provisions of the Raker Act, San Francisco is given the right to store water in the Hetch Hetchy Valley, Lake Eleanor, and Cherry Creek Reservoirs without any restrictions as to the order of development, as well as to construct and maintain power-houses, transmission lines, conduits, pressure tunnels, telephone and telegraph lines, and all appurtenances necessary for the operation of the municipal water-works and power supply. San Francisco must recognize the prior rights of the Modesto and Turlock Irrigation Districts to receive 2 350 sec-ft. when the same can be used beneficially to irrigate their lands and when that quantity of water is flowing in the Tuolumne River, provided said lands shall not exceed 300 000 acres; and San Francisco must further permit 3 000 sec-ft. of the natural daily river flow during the 60-day period immediately following and including April 15th of each year to pass its impounding dams, should this quantity ever be essential to the needs of the Irrigation Districts. Although the Raker Bill limits the irrigation obligations to 300 000 acres, the United States Army Board found that, with proper storage, there was ample for 400 000 000 gal. daily for San Francisco and for 400 000 acres of land.

In the Hetch Hetchy Valley, Lake Eleanor, and Cherry Creek Reservoirs, San Francisco can store economically 689 700 acre-ft. By utilizing other available storage sites within its drainage area, a total of 836 800 acre-ft. can be made available. According to the findings of the Board of Army Engineers* "Four hundred M. G. D. may be supplied San Francisco from the high areas without interfering with Turlock and Modesto rights, with 560 000 acre-feet of storage". The difference between 689 700 and 560 000 is the substantial excess of 129 700 acre-ft. which San Francisco would have as surplus storage after supplying all needs, by developing Hetch Hetchy, Eleanor, and Cherry.

In reservoirs outside the area controlled by San Francisco, the Turlock and Modesto Irrigation Districts can conserve 960 000 acre-ft.; 168 000 from its foot-hill reservoirs and 792 000 from the Upper Tuolumne. According to the report of the Army Engineers, previously referred to:

"The irrigation requirements of the Turlock and Modesto irrigation districts may be provided by their existing rights if conserved by means of 370 000 acre-feet of storage capacity."

* "Hetch Hetchy Valley: Report of Advisory Board of Army Engineers to the Secretary of the Interior, February 19th, 1913," p. 110.

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From these figures it is evident that in the Tuolumne River drainage area there is ample water to supply the demands of both San Francisco and the Irrigation Districts. Mr. Grunsky's doubts on this subject are no more scientifically justifiable than his assumptions on the invisible leakage from Crystal Springs Reservoir. Even were the danger of a shortage remotely possible, it must be remembered that, before exercising their priority to its limit, the irrigationists are required to develop their own local resources, and that it is expressly stipulated in the Act that the laws of the State of California relating to water shall in any event control as to the priority needs and use of water. By these State laws the City's interests are amply protected. Had no concessions been granted to the Irrigation Districts, San Francisco would not now enjoy a Federal Grant by which it is assured a permanent supply of 400 million gallons of pure mountain water daily. Moreover, as explained by the Army Board to the Committee on Public Lands, should a controversy ever arise between the City and the irrigationists as to their respective rights, domestic consumption, being the highest use to which water can be applied, under the laws of the State of California, the rights of the City will be paramount.

Mr. Grunsky regrets that:

"San Francisco, under the Raker Law, is to expend from $500 000 to $1 000 000 on the proposed storage works in excess of her own requirements, in order to be in a better position to meet the obligation, which she has apparently assumed, to supply some stored water at cost to the irrigation districts.

"San Francisco, [he says] under the same law, will be required to pay for the appropriated water, which should be hers without cost, under State law, at the rate of $15 000 for 10 years beginning in 1919, thereafter $20 000 per year for 10 years, and thereafter $30 000 per year."

Very few will agree with Mr. Grunsky that to make a capital outlay of from $500 000 to $1 000 000 for an increase of water supply from 60 million gallons to 400 million gallons daily is onerous, when the cost of developing the 60 million gallons daily would be approximately $45 000 000. The money paid by the City to the Government after 1919 is, according to the provisions of the Raker Bill "to be applied to the building and maintenance of roads and trails and other improvements in the Yosemite National Park, and other national parks in the State of California". San Francisco, being the commercial metropolis of California and the clearing house of the Pacific Coast, will naturally profit more by the expenditure of this money than any other city in the State. The prosperity of the Irrigation Districts and the enhancement of the attractiveness of the State's parks, which are visited yearly by thousands of Eastern tourists who make San Francisco their final destination, are of prime importance to the City, and

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money spent for these purposes will not be wasted. Certainly, it is not to be caviled at, in view of the enormous power concessions granted to San Francisco under the Raker Act.

In Mr. Grunsky's project power was to be developed sufficient merely to pump 60 000 000 gal. daily into San Francisco. The value of the power rights acquired with the enactment of the Raker Bill is conservatively estimated at $45 000 000, on the basis of the findings of the Army Board that 115 000 h.p. could be developed economically. Subsequent studies have disclosed the fact that 200 000 h.p. can be made available from San Francisco's Sierra sources. The percentage of this which will be used by the Irrigation Districts at cost is small. Mr. Grunsky believes that with oil at 75 cents per bbl. power can probably be developed more cheaply by steam in San Francisco than generated by water in the Sierras and transmitted a distance of 150 miles. The refutation of this theory is a matter of record in Case No. 400 before the Railroad Commission of California. In this case, in which the Pacific Gas and Electric Company was under investigation, both hydro-electric and steam plants were studied in order to determine a proper cost for electricity delivered at sub-stations for distribution. There are ten hydro-electric plants, and they include stations with capacities from 660 to 40 000 kw., having a combined capacity of 107 310 kw. There are four steam plants in the Pacific Gas and Electric system, totaling 70 200 kw. in output. Two of these plants have a combined capacity of 63 000 kw. in Curtis turbines. The capital cost of the hydro-electric plants is approximately $136.50 per kw., and the cost of the steam plants is $77.70. The average cost of energy deliverable at sub-stations from hydro-electric plants was $0.00488 per kw-hour. The average energy cost deliverable to transmission lines from steam plants was $0.01248 per kw-hour. These two costs include interest at 8%, depreciation, maintenance, and all operating expenses of generation. The cost of oil at the Oakland and San Francisco plants for the year considered was $0.68½ per bbl. To the cost for hydro-electric current should be added the cost for transmission and delivery to the sub-station. This amounted to $0.00177 per kw-hour. This brings the cost of hydro-electric current up to $0.00665 per kw-hour. These figures were made in July, 1914, for the purpose of fixing rates for the Town of Antioch. The State Railroad Commission has always resolved the doubt in favor of the corporation, when compiling data for the purpose of fixing rates, and has, no doubt, also allowed the Pacific Gas and Electric Company a considerable sum for capital expenditures made on property which has not as yet been developed to its full capacity. The developments considered in this report are all properties which were constructed primarily for the development of power, and are not plants from which power is merely a by-product, as in the Hetch Hetchy project,

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where the City must store water for domestic use, and can generate power with a very small additional expenditure. Mr. Grunsky's statement implies that all the electric companies which so recently have expended millions on hydro-electric development in the Sierras, have been guilty of gross misjudgment. If, as Mr. Grunsky states, the power feature is likely to prove a burden, then "the yoke is light and the burden sweet."

For an engineer to take issue on a point of law with attorneys and bonding experts of national reputation, who have passed on the validity of the $45 000 000 bond issue to construct the project approximately according to the Freeman plan, seems to be ill advised. It is likewise inconceivable that the people of San Francisco should object to this expenditure, to secure 400 million gallons daily, when they approved the bonds in the expectation of receiving only 60 million gallons daily.

Mr. Grunsky's innuendo, that the City Engineer has "a hand in the game", insinuating that politics is a governing feature in the conduct of the Bureau of Engineering is unjust, as is his statement that the natural results of Mr. Freeman's employment by San Francisco "was that thousands of dollars were needlessly expended". Every unprejudiced citizen of San Francisco, including many of those journalists who decried municipal ownership most energetically and fought hardest against the voting of bonds for the municipal system, agrees that the planning and construction of San Francisco's Municipal Railroads by the present City Engineer were accomplished with good judgment and efficiency. The phenomenal success of this enterprise would have been impossible had the City Engineer, to whom this success is in large measure due, been hampered by politics. The same applies to all of the City's engineering construction that has been undertaken within the past few years.

To prove his contention Mr. Grunsky states:

"Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone it is to be inferred that the political management of the city's water project will be continued."

By what intricate principles of logic Mr. Grunsky derives the conclusion from the premises is not apparent. When it becomes necessary for the chief engineer of a private corporation to request the directors for an appropriation for the temporary employment of one or more consulting engineers, he invariably names the engineers he desires to collaborate with him. Moreover, the personnel of the Board of Consulting Engineers precludes the idea of political preferment.

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It consisted of F. G. Baum, M. Am. Soc. C. E., retained as consulting engineer on almost every large hydro-electric development on the Pacific Coast and in South America; W. F. Durand, Professor of Mechanical Engineering at Stanford University and Consulting Engineer for Los Angeles power projects; and John D. Galloway, M. Am. Soc. C. E., who has had extensive experience in high-head power plants.

Continuing, Mr. Grunsky discusses the proposed purchase by the City of the properties of the Spring Valley Water Company. He states:

"According to the latest proposition, to be voted on some time next spring, the Water Company will be allowed to retain several thousand acres of valuable land in and adjoining San Francisco, besides some 5 000 acres of land in Livermore Valley. The value of the Pleasanton wells as a part of the system will be reduced because the maximum draft on them is to be limited to 15 000 000 gal. per day, and because the ownership of the 5 000 acres will carry with it the right to draw on the Livermore gravels for all the water that may be needed on these lands * * * there may not be much water left in the gravels for San Francisco."

The City Engineer of San Francisco was directed by the Board of Supervisors, in February, 1913, to designate such lands and properties of the Spring Valley Water Company as he considered it advisable for the City to acquire for use in connection with the early completion of the Hetch Hetchy project. After a very careful and comprehensive study he decided that of the lands owned by the Company at that time, amounting to more than 100 000 acres, it would be advisable for the City to acquire about 68 000 acres. The remaining lands comprised properties which are not desirable for use in connection with either the Spring Valley or the Hetch Hetchy system, some because the development of water from the sources in question would be uneconomical, some because the quality of the water is not satisfactory, and some because the right to divert water to the City would entail the acquisition of expensive riparian rights. In regard to the 5 000 acres of land in Livermore Valley excluded from the proposed purchase, the City Engineer only consented to the exclusion of these lands with the understanding that the City should have the right to take water from this region to the amount of, but not in excess of, 15 million gallons during any one day, this right being without hindrance or conditions of any sort, the Spring Valley Water Company waiving any and all rights to object to or enjoin the taking of such water, either on the ground that the pumping of underground water to the limit of 15 million gallons daily lowers the subterranean water plane under the lands retained by the Spring Valley Water Company, or that it drains the water beneath the retained lands, or on any other ground whatsoever. As 15 million gallons per

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day is the ultimate yield to which the Livermore gravels can be economically developed, the restriction of which Mr. Grunsky complains is not an onerous condition.

Mr. Grunsky states that the Hetch Hetchy project has not "yet received the necessary study to determine its main structural features." Enough study has been given the subject to determine definitely that the Grunsky scheme, involving many miles of open ditches, should be abandoned, and active construction has been started on the pressure system.

Mr. Grunsky's complaint of the compensatory features of the Bill was expressed by representatives of hydro-electric power companies in Washington.

There was written into the Hetch Hetchy Bill the conservation principles of the Roosevelt, Taft, and Wilson administrations, and the Federal compensation and regulation are phases of those conservation policies.

These questions were fully debated in the House and Senate, and in the House the vote was more than four to one for the Bill; in the Senate, two to one. Therefore, criticisms of these features are quarrels with the Federal Government policies, which, by the way, the City of San Francisco, so far as the Hetch Hetchy project is concerned, has approved unanimously.

Mr. Galloway.

J. D. GALLOWAY,* M. AM. SOC. C. E. (by letter).—As a part of his discussion of Mr. Cory's paper, Mr. Grunsky states the following:

"At the time the city thus voted bonds, there should have been created a Board of Water Supply, free from politics. This was not done. The administration at that time and the administrations that have followed, would not let go. The Mayor, who names the Board of Public Works, and who practically selects the City Engineer for the Board of Public Works, the Supervisors, with the Public Service and Finance Committees, the City Attorney, the Board of Public Works, and the City Engineer, all have a hand in the game.

"There is no programme as yet for any change. Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be for 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone, it is to be inferred that the political management of the city's water project will be continued. Deplorable as this may seem, it is yet possible that competent management will not be too greatly hampered by political obstruction and political direction; and the hope may be expressed that, despite the unfortunate circumstances of the past, San Francisco may yet achieve an adequate and in every way satisfactory water supply."

As one of the Board of Consulting Engineers on the power development, mentioned by Mr. Grunsky, the writer desires to make the

* San Francisco, Cal.

following explanation, in order that the apparent strictures on the Board by Mr. Grunsky may be understood. Mr. Galloway.

The definite plan of a water supply, on which was based the bill granting the right of San Francisco to use the Hetch Hetchy Valley as a storage reservoir, was made by John R. Freeman, M. Am. Soc. C. E., in a very comprehensive report on the entire situation. A clause in the bill required that within a certain definite time the City should file with the proper department at Washington maps showing the definite location of the conduits to be used, so far as they crossed Government lands. Mr. Freeman's report was very complete as a whole, but, necessarily, the details, in part, required further consideration. This was particularly so in the case of the development of power as an adjunct of the water supply. Mr. Freeman's suggested design had in view the use of a tunnel for the conduit, leading to a point above one of the proposed power-houses, and thence a steep pressure tunnel in the rock down to the power-house, under a head of about 1 300 ft. As a means of controlling the possible surges in the water conduits, Mr. Freeman had suggested a surge chamber at the junction of the conduit tunnel and the pressure tunnel to the power-house.

It is a well-known fact that though surge chambers of proper dimensions and shape will provide for proper speed regulation of hydro-electric units, they do not in any way provide for taking care of the daily variations in power demand, designated as load factor. The flow of the water in the conduit being practically uniform, the maximum demand on the power-station is limited to the power which can be supplied by the maximum flow of the conduit. If, on the other hand, it is possible to supply at the lower end of the conduit, at a point just above the power-station, a reservoir of sufficient size, the daily fluctuations in power demand can be supplied therefrom, the uniform flow of the conduit passing at all times into the regulating reservoir. It is a rare thing for any power system to have a load factor of much more than 50%, and a number of the important systems of the country have load factors of less than this. In Mr. Freeman's report, the power development was considered as incidental to the water supply. The conduit was considered primarily as one which was to bring water to San Francisco, and the peak load of the power-plant was limited to the maximum capacity of the aqueduct tunnel, more than 19 miles long.

In requesting the Board of Consulting Engineers to serve, Mr. O'Shaughnessy stated that he was confronted with the necessity of presenting maps showing the final location of the conduits, within a limited time; and he also stated that a further study should be made of the power-station design, in order to see if the power development could not be made so as to obtain the maximum development of power, having in view a possible load factor, and without interfering at all

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with the flow of the water. To answer the various questions asked by the City Engineer really required, on the part of the Board of Consulting Engineers, a complete study of the entire situation of the development of power at the lower plant of Mr. Freeman's design. It is not necessary to dwell on the results of the study as made.

The writer wishes to make clear this point, that the City Engineer was under the necessity of determining these several features before a given definite date. Some appreciation of the problem can be gained when it is said that on a load factor of 50%, the station would generate a peak load of about 100 000 kw. The Board of Consulting Engineers could not answer the questions without a complete study of the situation, and the City Engineer was hardly justified in determining the final location of the project until it had been examined.

To one who reads the statements made by Mr. Grunsky, it may appear that politics formed the principal element in the appointment of the Board of Consulting Engineers. In order that the readers of this discussion may form their own opinions, the following information is given: The Board was formed of Professor W. F. Durand, head of the Department of Mechanical Engineering at Stanford University, Mr. F. G. Baum, and the writer, both Members of this Society, in independent practice in San Francisco. Professor Durand, in addition to his duties at the University, has had considerable experience in hydro-electric work. He has made a thorough study of the problems of surge chambers, and was connected with the design of the surge chamber in the power-plant of the Los Angeles Aqueduct. As the surge chamber suggested by Mr. Freeman was an important point in the review, it was felt that Professor Durand's services were especially valuable in this as well as in the other phases of the subject. Mr. Baum has been connected with hydro-electric work in California for nearly twenty years. He was employed on the design of the first long-distance transmission line in the State, with the Standard Electric Company. He was for years the General Superintendent of the Pacific Gas and Electric Corporation, having in charge the building and operation of the numerous plants of that system, which is the largest in California. For a number of years he has acted as the Consulting Engineer of that Company on some of the largest hydro-electric installations in the West. The writer has been connected with the design and construction of hydro-electric works for fifteen years, has acted as Consulting Civil Engineer for the Pacific Gas and Electric Corporation for several years, was a member of the Board of Advisory Engineers on the Sierra and San Francisco Power Company installation, has had charge of the construction of several smaller plants, and recently acted as Consulting Engineer on the installation of the fifth unit, of 10 000 kw. capacity, of the Great Western Power Company. None of the members of the Board is a resident of the City of San

Francisco, and all are without connection politically with any one in the city. There would seem to be nothing improper in the fact that the three members of this Board were named by the City Engineer in the ordinance authorizing their employment. Most engineers will agree that it was better for the City Engineer to make the selection than to leave it to the non-technical Supervisors or the Board of Public Works. As a matter of fact, no member of the Board ever conferred with any member of the City Government except the City Engineer. It would also seem that the previous experience of those named sufficiently justified the judgment of the City Engineer in his selections, and that the appointments were as far removed from the domain of politics as it is possible to have them. Mr. Galloway.

In Mr. Grunsky's statements, the imputation is also conveyed that the various officials of the City refuse to provide for a Board of Water Supply because they all desire to "have a hand in the game". The writer is not acquainted with the opinions of any members of the City Administration, except those of the City Engineer, Mr. O'Shaughnessy, and he has stated publicly, before the Commonwealth Club of San Francisco, and also has expressed the opinion to the writer a number of times, that the proper way to take care of the construction of the Hetch Hetchy project would be to place it entirely in the hands of a Board of Water Supply disconnected from the other departments of the City's organization. Apparently, the whole procedure is at present awaiting the election for the purchase of the local water company's plant. It is quite possible that when this subject is decided, steps may be taken looking to the formation of a Board of Water Supply, but of this the writer has no knowledge. It would seem that some recognition of the public utterances of the City Engineer on this subject should be taken as an evidence of his opinion in the matter.

The writer has been a firm believer in the project of bringing water from the Sierra Nevada Mountains to the Bay districts of San Francisco, which was first definitely set forth by Mr. Grunsky. That the project has suffered various changes since the original plans were proposed, due largely to enlargement of the idea, is only natural. It is believed by the writer that the report by Mr. Freeman was an able and comprehensive document which considered thoroughly all the needs of the situation as it then appeared. Full credit must also be given to Mr. Grunsky and Mr. Manson for their earlier labors along the same line. Mr. Freeman's report was undoubtedly of material effect in securing for the city the rights for which it was struggling. The application for permits under earlier administrations had failed, and it was necessary for the officials in charge to make such compromises as they deemed expedient at the time when the Bill was finally passed by Congress. The conditions became a matter

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of judgment, and those who were in charge must be given credit for honesty of intentions and a knowledge of the situation as it then existed, even though the conclusion reached may differ from the ideas held by others.

Mr. Baum.

F. G. Baum,* M. Am. Soc. C. E. (by letter).—The writer notes the following statement† by C. E. Grunsky, M. Am. Soc. C. E.:

"At the time the city thus voted bonds, there should have been created a Board of Water Supply, free from politics. This was not done. The administration at that time and the administrations that have followed, would not let go. The Mayor, who names the Board of Public Works, and who practically selects the City Engineer for the Board of Public Works, the Supervisors, with the Public Service and Finance Committees, the City Attorney, the Board of Public Works, and the City Engineer, all have a hand in the game.

"There is no programme as yet for any change. Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be for 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone, it is to be inferred that the political management of the city's water project will be continued. Deplorable as this may seem, it is yet possible that competent management will not be too greatly hampered by political obstruction and political direction; and the hope may be expressed that, despite the unfortunate circumstances of the past, San Francisco may yet achieve an adequate and in every way satisfactory water supply."

The fact that the City Engineer did select the men and the further fact that none of them is a politician (and does not even reside in San Francisco) should both be evidence that politics had nothing to do with the appointments. Mr. Grunsky's remark is like stating that two forces acting in the same direction could produce motion in a body in an opposite direction, both his premises being opposed to his conclusion.

The fact that there was immediate necessity for directing certain matters specifically, which had been only dealt with in general by Mr. John R. Freeman, in order to file the proper papers with the U. S. Government, was the reason for the appointments.

Mr. Grunsky's premises, his conclusions, and his inferences are therefore wrong.

Mr. Hopson.

E. G. Hopson,‡ M. Am. Soc. C. E. (by letter).—The discussion of this paper by the City Engineer of San Francisco reveals a point of view of decided interest and importance, being, in fact, the official viewpoint of the situation as it is to-day. Mr. O'Shaughnessy, who

* San Francisco, Cal.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

‡ Portland, Ore.

Mr. Hopson.

is comparatively a newcomer in official circles, undertakes to defend in a somewhat chivalrous manner many of the lapses of his predecessors by endeavoring to reconcile these with the present status and to persuade us that the action now proposed in certain official circles is the best.

Mr. O'Shaughnessy complains that Mr. Cory's paper "consists chiefly of a résumé of matters which were thoroughly and scientifically discussed and settled"; also that "Mr. Cory attempts to revive a number of dead issues, and questions the engineering and financial policies of San Francisco as if these matters had not already been thoroughly considered and disposed of by a number of distinguished engineers", etc. This is the crux of the entire matter, and the writer can well understand that it is at least embarrassing to the official mind to find that, even after all the elaborate investigations that have been made during the past few years and the momentous decisions that have been reached by past officialdom, the main question at issue still refuses to be cavalierly disposed of, but confronts us as if nothing had been done nor any investigations or reports had ever been made.

Thus, after all that has happened, the real issue forces its way to the front in all its baldness and crudity: Does San Francisco want any distant supply? Is the Tuolumne supply what it wants? To these questions may now be well added the minor one as to whether it is not cheaper and better to lose even whatever moneys have been spent on the Tuolumne system to date, than spend forty or sixty good dollars to save one bad one. The last issue has been created by the well-meaning enthusiasts who have led the City to commit itself to expenditures on the Tuolumne supply to an extent amounting, the writer is informed, to more than $1 000 000.

Among the smoke and dust of political and engineering controversy on this classical topic, the following points can be discerned as solid ground, affording certain though limited support. These points are:

(*a*) The practical certainty that the Spring Valley system can be acquired, if the City so desires, at a price that has now been mutually agreed to both by the City and the Company. This is a highly important feature.

(*b*) That the Spring Valley system is susceptible of developing a capacity of between 100 000 000 and 200 000 000 gal. per day, according to the authority that may be preferred. It is important to have in mind that these capacities are, respectively, about double and quadruple the present uses of water in the City.

(*c*) That if the City should acquire the Spring Valley system it would have a municipal supply sufficient to last it for the next 20, or possibly the next 50, years.

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(*d*) That the quality of the Spring Valley water is admitted to be excellent.

It may be logically deduced from the foregoing axioms, therefore:

1.—That the objection of the people of San Francisco to a privately owned water supply can be immediately removed by the acquisition of the Spring Valley system;

2.—That there is ample time to consider and settle all the vexed engineering questions as to the available supply in the present system and the City's needs, long before any additional supply to the present sources will be required.

In the light of the foregoing, does it seem reasonable for the City to proceed at this time to commit itself to any plan leading to expenditures ranging from $37 000 000, as estimated by Mr. Freeman, to $64 000 000, as estimated by Mr. Cory and others, on an independent system that is certainly not needed to-day nor to-morrow, nor for a great many years, and which, moreover, appears to a number of apparently competent engineers and business men to be not well advised?

Mr. Cory and other thinkers of San Francisco urge that the City should pause to consider and weigh carefully the issue and not rush precipitately into these vast expenditures or do anything further to embarrass the community, already heavily encumbered, with another load of debt. They believe there is no reason for action on the Tuolumne or any other additional supply for a long time and that the City cannot afford to commit itself either to a $37 000 000 or to a $64 000 000 proposition, much less mistake. It is urged, and the writer believes properly, that unnecessary expenditures, which are tantamount to waste, in this direction, must be reflected by curtailments in city improvements along other much more important lines.

Take, for instance, the proposition as it is advanced to-day by the advocates of the Tuolumne scheme—the City is to proceed of its own responsibility to build a metropolitan water supply system intended to serve not only San Francisco but all the bay cities. Did the City of Boston proceed to build the Metropolitan Water System of Massachusetts of its own initiative before the Massachusetts Metropolitan Water District was created, on the expectation that all the other metropolitan cities would enter at some future date and share the cost? Not that the writer is aware. Did the City of New York adopt a Metropolitan Supply sufficient for Brooklyn, Staten Island, and other communities around the mouth of the Hudson before the City was consolidated into Greater New York and all members of the metropolitan district were established as component parts of the system? It did not.

Mr. Hopson.

Is there any known case, worthy of being cited as an example, where a metropolitan water system has been built ahead, or independently, of the legal subordination of the several parts of the system to the whole? What would any one think of the business acumen of an individual who, living in a village community, undertook, not only to lay a pipe to supply his own house and barn with water from the near-by spring, but mortgaged his farm to pay for a system of pipes to supply all his neighbors, without taking the trouble to consult them seriously or to make any agreement with them in advance? It does not require much imagination to picture the troubles and embarrassments he would encounter in trying to pay interest on his borrowed money, year after year, or his entreaties to his neighbors that they hurry up and take his water and relieve him of the crushing burden he had assumed so rashly on their account. The proponents of precipitate action on the Tuolumne supply matter are endeavoring to place the City of San Francisco in this invidious position.

The question at issue is not wholly one of cost, or the difference between a $37 000 000 Freeman scheme or one of $64 000 000, as estimated by others; nor does it consist of the difference between the estimated capacity of the Spring Valley system, as to whether it can be made to yield 100 000 000 gal., as urged by Mr. O'Shaughnessy, or 200 000 000 gal., as claimed by General Chittenden and others. The real question at issue is whether the Tuolumne proposition is intrinsically a good one, and, even if so, whether it is a good business proposition from the standpoint of the City.

This question, in the writer's judgment, is entirely unsettled, with the preponderance of evidence thus far presented against the Tuolumne scheme. The essential point, however, is that no action should be taken to commit the City to this scheme or any other scheme for at least 5 years, probably not for 10 years, after the Spring Valley system shall have been acquired, and in no event until a Metropolitan District shall have been legally organized.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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PENSTOCK AND SURGE-TANK PROBLEMS

Discussion.*

By H. C. Vensano, M. Am. Soc. C. E.

H. C. Vensano,† M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest, but disagrees with one of the author's assumptions and its resulting formula, and calls attention to a possible improvement in his formula for the economical size of penstock. Mr. Vensano.

Considering this latter first, attention is called to the fact that his formula, $D = \sqrt[6]{\frac{3\,250\,q^2\,i}{C^2\,p\,B}}$, includes B which, as defined by him, is "Cost of pipe in place, per foot of diameter per foot of length", is in itself a variable in D. The cost of metal conduits per foot of diameter is dependent on the weight of metal in the pipe. This depends on and is directly proportional to the thickness of the pipe, which in itself is directly proportional to the diameter. (If wood stave pipe is being considered, that portion of the cost represented by the bands will follow the same law.) In general, then, B is directly proportional to D, and Equation 44 can be written with its second term as a function of D^2, bringing the final expression to the form, $D = \sqrt[7]{\frac{3\,250\,q^2\,i}{C^2\,p\,2\,Z}}$, where constants are as assumed by Mr. Warren, and where B has been taken equal to $Z\,D$; Z being a constant.

This formula in the past has been proved in a little different form in connection with the discussion of a paper by the late Arthur L. Adams, M. Am. Soc. C. E., entitled "A Solution of the Problem of Determining the Economic Size of Pipe for High-Pressure Water-

* Discussion of the paper by Minton M. Warren, Jun. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† San Francisco, Cal.

Mr. Vensano.

Power Installation".* In this paper Mr. Adams arrived at the theorem that:

"That pipe fulfills the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals four-tenths (0.4) of the annual cost of the pipe line."

In the discussion of this paper by the late James H. Wise, Assoc. M. Am. Soc. C. E., and by Charles D. Marx, President, Am. Soc. C. E., and C. B. Wing, M. Am. Soc. C. E., this theorem was proved mathematically, and the resulting formulas can easily be changed to a form containing the seventh root of D. The formula in such form is very practical when considered with metal pipes, in that a pound price for "metal in place" may be used in it, rather than a price "per foot of pipe per foot of diameter." The price of metal per pound can be assumed closely for any given installation, whereas the price per foot of diameter, as already pointed out, varies with the diameter to be determined, and therefore is uncertain and difficult to fix.

Referring now to the formula developed for ordinary water-hammer, Mr. Warren writes:

"If the gate continues closing, however, a new wave and corresponding rise of pressure will be started for each increment of the gate motion, and as long as the gate moves the pressure will continue to rise until the reflected waves interfere. It is assumed that the gate is closed in such a way that, in the absence of reflected waves, the pressure will rise at a constant rate. When the waves come back, the rate at which they are diminishing the pressure is the same as that at which the gate is increasing it. (See Fig. 2.) The pressure, therefore, will become constant until the gate is stopped, when it will begin to fall again, reaching normal after a time, $\frac{2L}{a}$. This is the first assumption, and is very near to the truth as experiments have shown."

Although the first two sentences of this assumption are no doubt allowable, and in fact in general are made by most investigators of this subject, the rest would not follow. Instead of the pressure rising steadily to a maximum and becoming constant until the gate is stopped, it should, following out the assumption of partial pressure wave returns, rise in a series of waves, reaching a definite maximum and then again falling to normal, but never passing to sub-normal until the gate is completely closed. The reason for this is that when a partial wave returns to the gate it not only returns to normal, thus neutralizing its own increment of pressure, but passes through normal to sub-normal, and, during the time of its second traverse of the pipe, counteracts in addition the new pressure increment, which is just

* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 173.

leaving the gate, thus producing a gradual return of the total pressure in the pipe at the gate to normal. Mr. Vensano.

On the completion of one entire cycle in time, $\frac{4L}{a}$, each partial pressure wave again passes through normal to super-normal, and the total pressure in the pipe is gradually again brought up to its full value.

On such hypothesis, the water-hammer pressure at the gate during the time of gate closure (always provided the time of gate closure is longer than $\frac{4L}{a}$), would rise to a maximum a number of times, but would never, however, as stated previously, pass below normal until after the gate is completely closed. At a later time, of course, it would pass from super-normal to sub-normal in cycles, gradually dying out as the pipe friction reduced the wave effects.

To illustrate this, the writer submits Fig. 7, a diagram made up along the lines of those of Joukovsky, or at least as used by O. Simin in the translation of his work. Each line represents a partial pressure wave due to a partial gate closure. The point of start of each such line is, of course, at normal pressure. The stepped line at the bottom of the diagram gives the result of superimposition of all the partial pressure lines above. This diagram is made up for an assumed time of gate closure of 40 sec. and a length of pipe line (quite long) such that $\frac{2L}{A} = 10$ sec. It has only been drawn for the 40-sec. period, or the time of duration of gate closure, and has not been carried to the point where the waves would pass below normal. These waves are for a point indefinitely close to or at the gate. The sloping line, *A B C D E*, would be the resulting pressure line if infinitesimal pressure increments had been used.

Mr. Warren's diagrams showing a constant rise during the time interval, $\frac{2L}{a}$, a constant pressure during the time interval, $\frac{T - 2L}{a}$, and a drop in pressure during the final period, $\frac{2L}{a}$, is therefore incorrect, which would apparently make his resulting formula likewise incorrect.

In substantiation of the contention that the assumption of constant pressure during gate opening is incorrect, the writer submits the following: The Pacific Gas and Electric Company, with which the writer is at present connected, completed in 1913 its Drum Hydro-Electric Plant. In connection with this station, a riveted penstock line about 6 300 ft. long and varying from 1¼ in. in thickness and 52 in. in diameter at the power-house to 72 in. in diameter and ¼ in. in thickness at its upper end, was built. The work was completed in November, and the writer

Mr. Vensano.

attempted to make some tests on the pressure effects in the penstock due to opening and closing the power-house nozzles.

The flow of water at Drum is controlled by motor-operated needle-nozzles, which give an approximate linear decrease and increase of velocity of flow in the pipe. The time of closure was 75 sec.

Unfortunately, the weather was very cold at the time, and conditions were such as to interfere with the gauges by freezing. On this account, the results were rather unsatisfactory, and the tests were at that time abandoned. As the plant, since then, has been in operation continuously, it has not been possible to do anything further. It is hoped that these tests may be resumed in the near future.

The results obtained, however, showed that during the time of opening or closing the nozzles there were a number of rises and falls of pressure, and that the pressure did not remain constant during the entire period of operating the gate. The endeavor was made to take pressure readings at three points varying from 3 000 to 6 000 ft. from the power-house, while one or more nozzles in the station were opened and closed, both separately and together. Unfortunately, all the gauges, except one about 4 000 ft. from the station, were in exposed positions, and, because of the freezing of these two gauges, the corresponding results for different points on the line could not be obtained.

The writer suggests the general formula,

$$h = \frac{2\,l\,V}{g\,T} \qquad (1)$$

for the maximum pressure at any point in the pipe line and for any time of gate closure, with the limitation, however, that h can never be greater than $\frac{V\,a}{g}$, and that, when the formula gives greater values, h will equal the limiting value, $\frac{V\,a}{g}$. In this formula, h is the excess pressure over normal (not static) due to water-hammer, and l is the length of pipe, measured from the reservoir or source of supply to the point at which the pressure is desired. All other symbols are used as defined by Mr. Warren.

To obtain this formula: Consider a short time of gate closure and an indefinitely long pipe line, so that no waves return from the reservoir before the gate is completely closed. Following the usual assumption that,

"If the gate continues closing, * * * a new wave and corresponding rise of pressure will be started for each increment of the gate motion, and as long as the gate moves the pressure will continue to rise until the reflected waves interfere," and further, "that the gate is closed in such a way that, in the absence of reflected waves, the pressure will rise at a constant rate,"

Mr. Vensano.

PRESSURE DIAGRAM FOR POINT NO. 0
Gate
PIPE
Reservoir
0 1 2 3 4 5
DURATION OF CLOSURE, 40 SEC.
$\frac{2L}{A}$ Assumed = 10 Sec.
Where L = Length of Pipe
and A = Velocity of Pressure Wave along Pipe
Pressure
A
B
C
D
E
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40
Time, in Seconds.

FIG. 7.

Mr. Vensano.

we obtain a rise of pressure at the gate represented by a straight line of constant slope. The total pressure rise, H, provided there is no interference, will be equal to $\frac{V a}{g}$, as given by Joukovsky and substantiated by Mr. Warren. The slope of the line representing this pressure rise will then be

$$\frac{H}{T} = \frac{V a}{g t},$$

H, as used hereafter, is the absolute maximum value of h, and is equal to $\frac{V a}{g}$.

If this rate holds for instantaneous closure, it will likewise hold for each partial instantaneous increment of closure of a more slowly moving gate; and, as the closure is assumed to be continuous, it should hold for all values of T.

Now consider the phenomena occurring at any point, X, whatsoever in a pipe line. If the main gate is assumed to close instantaneously, causing a complete stoppage of the water column at the gate and a compression wave to travel up the pipe, note that at the point, X, all conditions of velocity and pressure will remain the same until the pressure wave reaches this point, at which time the result at X is exactly as though a gate were instantaneously closed at this point (X) in the pipe line, for the reason that, since the water below this point has been compressed to its maximum, no further compression can take place, and the velocity in the line is here instantly changed to 0. The first pressure rise at the point, X, therefore, will be at the same rate, and will reach the same maximum, before interference occurs, as though a gate were suddenly closed at this point. (The character and duration of further waves, of course, will be different, because returning waves are not here reflected as at a gate, but pass through the imaginary gate.)

Similarly, for continuous gate closure in a manner as assumed, each small instantaneous increment of closure would have the same effect on the first pressure rise at the point, X, as it had at the gate, and the total of all such increments would likewise give results as at the gate until such time as interference from return waves occurs. Therefore, for any point in the pipe line, and for all values of T, the rate of first rise in pressure may be taken at $\frac{H}{T}$. Then at any point, X, the first interfering wave will return to this point at a time, $t = \frac{2\,l}{a}$, after the imaginary gate at this point has closed. The imaginary gate at X necessarily closes in the same time, T, as the real gate at the foot of the line, as the lapse of time between the

instant that the first partial pressure wave passes X and the time that the last partial pressure wave reaches X and builds up the maximum pressure is T. Mr. Vensano.

The value of h at which the pressure rise is cut short at any point, therefore, will be given by the proportion $h : H :: \frac{2\,l}{a} : T$

or $$H = \frac{a\,V}{g},$$

therefore, $$h = \frac{2\,l\,V}{g\,T}.$$

Thus the general formula suggested is obtained. As h can never be greater than $\frac{a\,V}{g}$, for all values of l which give greater results, h must equal $\frac{a\,V}{g}$. The first (nearest the origin or reservoir) point, hereafter called the "critical point," at which the formula gives the maximum value for the pressure, will be that point for which

$$h = \frac{a\,V}{g} = \frac{2\,l\,V}{g\,T},$$

or $$l = \frac{a\,T}{2}.$$

For all points between this critical point and the gate, the formula will give values of h in excess of $\frac{a\,V}{g}$, and the maximum pressure will obtain, (See Fig. 9 (*c*).) Between the origin and the critical point, the co-ordinates of which will be called h_y and l_y, the equation is strictly applicable, and $\frac{h}{l} = \frac{2\,V}{g\,T}$, which is a constant for any fixed set of conditions.

Therefore we may write $$\frac{h}{l} = \frac{h_x}{l_x} = \frac{h_y}{l_y}.$$

Joukovsky arrived at this same result by a study of pressure-wave diagrams which he constructed for various points. Fig. 8 is such a diagram; it has been drawn for the same assumed conditions as Fig. 7, and shows graphically the pressure waves during the time of gate closure at a point three-fifths of the length of the pipe from the gate, or two-fifths of the distance from the reservoir.

By comparison of this diagram with Fig. 7, it can be seen that, for the case here represented, the maximum pressure, P', at this point is equal to 0.4 of the maximum pressure, P, at the gate, or in the ratio of $\frac{2}{5}\,L$ to L. From such study he formulated the conclusion that the ratio of the pressure at any point to the maximum pressure, is equal to the ratio of the time required for the wave to make a trip from the point in question to the reservoir and return to such point, to

Mr. Vensano.

PRESSURE DIAGRAM FOR POINT NO. 3
Gate
0 1 2 3 4 5
Reservoir
PIPE
DURATION OF CLOSURE,
40 SEC.
$\frac{2L}{A}$ Assumed = 10 Sec.
Where L = Length of Pipe
and A = Velocity of Pressure
Wave along Pipe
Pressure
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40
Time, in Seconds.

FIG. 8.

the time required to make a trip from the last (first where measured from the origin or reservoir) point of maximum pressure to the reservoir and return to such point. This conclusion he expressed by the formula,

Mr. Vensano.

$$\frac{P_x}{P} = \frac{t_x}{t},$$

which may be easily changed to the form,

$$\frac{h}{l} = \frac{h_y}{l_y}, \text{ as obtained previously.}$$

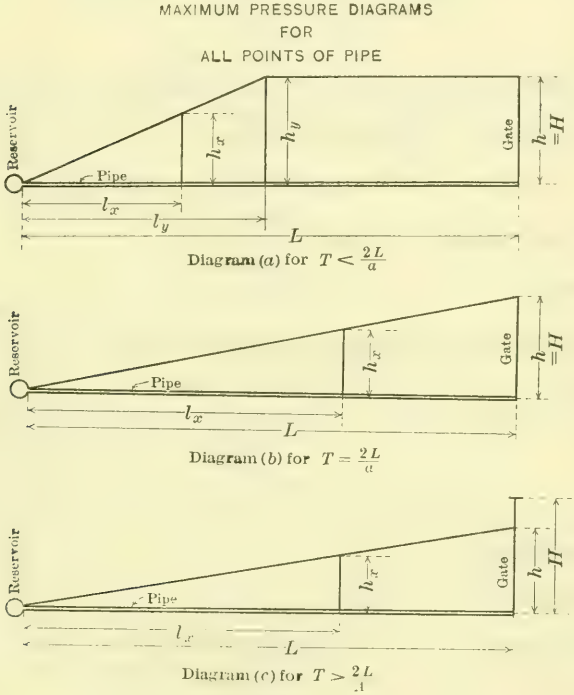

FIG. 9.

To show the general applicability of the proposed formula, the diagrams, Fig. 9 (*a*), (*b*), and (*c*), have been drawn for the three conditions, respectively,

$$T < \frac{2L}{a},$$

$$T = \frac{2L}{a},$$

and $$T > \frac{2L}{a}.$$

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It is to be noted that, for times of gate closure greater than $\frac{2\,L}{a}$, the critical point is an imaginary one, and occurs on an imaginary extension of the pipe line beyond the gate.

Case I.—Instantaneous Closure.—For this case, $T = 0$.

$$h = \frac{2\,l\,V}{g\,T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

substituting $T = 0$, $h = \infty$. This holds for all points in the line, but, by limitation, h cannot be greater than $\frac{a\,V}{g}$. Therefore

$$h = \frac{a\,V}{g}$$

for all points of the pipe, as it should for instantaneous closure.

Case II.—(See Fig. 9 (a)).—$T < \frac{2\,L}{a}$

$$h = \frac{2\,l\,V}{g\,t},$$

for a point at the gate, $l = L$ and $h = H$.

By substitution, $$h = \frac{2\,L\,V}{g\,T}.$$

By the limitation that h can never be greater than $\frac{a\,V}{g}$, and, as in the last formula,

$$T < \frac{2\,L}{a},$$

therefore $$h > \frac{a\,V}{g},$$

and, by the limitation, $$H = \frac{a\,V}{g}.$$

For the critical point, $$h = h_y,\ l = l_y,$$

as already shown, $$l_y = \frac{a\,T}{2}.$$

Substituting, $$h = \frac{2\,l\,V}{g\,T},$$

$$h_y = \frac{a\,V}{g}.$$

For any point between these two, the formula gives results greater than $\frac{a\,V}{g}$, and, by limitation, the absolute maximum pressure, $\frac{a\,V}{g}$, occurs at all points in this portion of the line. For all points between this critical point and the reservoir, the equation gives values decreasing uniformly to zero at the reservoir.

Mr. Vensano.

Case III.—(*See Fig. 9 (b)*).—$T = \dfrac{2\,L}{a}$,

$$h = \frac{2\,l\,V}{g\,t},$$

at the gate $l = L$,

$h = H$,

substituting the value of T, we obtain

$$h = H = \frac{\dfrac{2\,L\,V}{g}}{\dfrac{2\,L}{a}} = \frac{a\,V}{g}.$$

as it should be. For all other points, values decreasing from this maximum to zero at the reservoir will be obtained.

Case IV.—(*See Fig. 9 (c)*).—$T > \dfrac{2\,L}{a}$,

$$h = \frac{2\,l\,V}{g\,T},$$

at the gate $l = L$,

and $$h = \frac{2\,L\,V}{g\,T}.$$

For all other points, values uniformly decreasing from this maximum to zero at the reservoir will be obtained.

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DEPRECIATION AS AN ELEMENT FOR CONSIDERATION IN THE APPRAISAL OF PUBLIC SERVICE PROPERTIES

Discussion.*

By Messrs. Clinton S. Burns, Halbert P. Gillette, and W. Kiersted.

Mr. Burns.

Clinton S. Burns,† M. Am. Soc. C. E. (by letter).—The writer is particularly interested in expert valuation work and appraisal of public service properties, and is glad to express his appreciation of this valuable and interesting paper.

In his discussion of a paper‡ by John W. Alvord, M. Am. Soc. C. E., the writer presented his views on this subject somewhat in detail, and will now merely amplify certain phases of that discussion.

If the expert appraiser could place himself in a position of absolute impartiality in the consideration of this question, so that his conclusions would be entirely unbiased by any consideration of favoritism toward his client, he should then feel satisfied with the results of his analysis, and the writer would suggest that this result could be obtained by assuming, for the purpose of this discussion, that there are no private interests at stake and that the property being valued is municipally owned, having been built by a bond issue bearing 6% interest and having been operated heretofore (as most municipal plants are) without any particular reference to proper rates or adequate returns. Now there comes a business-like municipal administration requiring that an appraisal be made of this municipal plant, together with an investigation of its revenues, this appraisal to be used

* Discussion of the paper by C. E. Grunsky, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Kansas City, Mo.

‡ "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 842.

Mr. Burns.

as a basis for a revision of the rates, the object being that the new rates shall produce adequate revenues to make the property self-supporting.

Let it be assumed that the appraiser finds the property described in fairly good operating condition, and proceeds to determine how much operating revenue must be collected in order to preserve the property in satisfactory operating condition and perpetuate the utility. It is obvious that the fundamental principles involved in such an investigation are entirely independent of whether the property is owned by a municipality, by an individual, or by a corporation, and these principles are the same for a short-lived property as for one of long life; and it may simplify the problem and clarify the analysis if it be assumed that the property is made up entirely of perishable items, the entire property having the same average life or expectancy as that for which the original construction bonds were issued. In these assumptions, it is evident that the property is made just self-supporting when the revenues are sufficient to pay all operating expenses—6% interest on the bond issue and a sufficient sum to renew the plant when the original bonds become due, this being the expectancy of its life. It is evident, therefore, that the renewal fund, or replacement fund as it is termed by the author, must be sufficient to retire the original bond issue plus whatever sum may have to be accumulated to pay for the increased cost of renewal of the property at the expiration of its life, in case such renewal costs more at that time than the original bond issue; or, conversely, minus whatever sum the renewal may cost less than the original bond issue. The fund that will retire the original bond issue, plus or minus the corrections necessary to care for changed conditions in the cost of renewal, therefore, will represent the proper replacement fund in this instance. This view of the problem makes it plain that, in this case, no matter what the present value or the depreciated value of the property may be, the rates must be based on the future cost of reproduction, else, when the time comes that replacement is necessary, it will be found that the revenues have been inadequate to provide for replacement and, therefore, the property has not been self-sustained. In the case of a municipal plant, as assumed in this analysis, the emergency is met by a larger bond issue, but this is not an equitable solution, because it is discriminating against the future generation in favor of the present.

This analysis also throws light on the equity of including the cost of cutting and replacing existing street pavements, where the object of the appraisal is to determine a basis for rate-making. It has been the practice of many State Public Service Commissions to adhere in part to the theory of the cost of reproduction of a property, but to eliminate from such reproduction cost such items as the added

Mr. Burns.

cost due to street pavements, on the theory that this added cost, not having been borne by the utility and therefore not constituting a portion of its investment, is properly eliminated. The fallacy of this argument has been pointed out so frequently and so forcibly by the leading expert appraisers that it may seem superfluous to call attention to it again in this discussion, but the persistency with which some of the State Commissions adhere to these fallacies is sufficient excuse for the reiteration.

The fallacy is that such a procedure confuses present value with past cost. However, irrespective of that fact, it occurs to the writer that all existing pavements must be included in any valuation that is to be used as a basis for rate-making, because this determines to what extent the owner of the utility will be compelled to provide funds with which to restore the pavements when he is actually confronted with the question of renewing the obsolete or decayed pipes. The following will perhaps emphasize this truth.

A public utility, as stated by the author, has unlimited life; it must be perpetuated; therefore, every perishable part must be renewed by the owner whenever such part becomes incapacitated for its work. Every pipe line must ultimately meet this fate. The revenue must be sufficient to pay all operating expenses, interest on the investment, together with an additional sum sufficient to perpetuate the property. To perpetuate means not only to provide for the ordinary maintenance, repairs, and up-keep, but to guarantee the requisite replacement fund as well. Now, every street that is paved necessarily increases the owner's expenses in renewing the pipes, and such renewal is a liability which he must ultimately meet face to face. Therefore, if present rates are based on a valuation that does not include the element of enhanced value accruing from the item of street pavements, then the revenue is deficient and confiscatory by just that amount. Similarly, all metropolitan influences, such as growth of traffic, interference of sewers, telephone conduits, underground electric systems, police regulation, street railways, etc., each of which tends to increase the difficulty and cost of renewal of structures—particularly of pipe systems and other underground structures—must be included in any valuation to be used as a basis for rate regulation, for if any of these be not thus included, then, when the time comes for replacing the old structures, additional capital must be incorporated in the property, with no corresponding betterment in the construction account.

Concerning the author's statement that "the annual replacement increment should ordinarily be assumed to be earning interest at the same rate as the entire business. * * *, if the same be invested in large part in the business", the writer wishes to direct attention to the fact that such re-investment of the annual increments of the replacement fund entitles it to a higher rate of interest than is justi-

Mr. Burns.

fiable in computing the sinking-fund accrument; or, to state it another way, the 7% rate of return, earned by the water-works or gas plant referred to by the author, is properly considered as not only the earnings of the invested capital, but includes also the owner's risk, his reward for industry, his time devoted to the investment, and all the other elements which go to make up the difference between liquid and invested capital. It seems to the writer, therefore, that the proper rate of interest to be used in all sinking-fund computations is the current rate for liquid capital, regardless of how much the invested capital may be earning.

The author states that the correctness of any method of appraisal for rate-fixing purposes is dependent on its continuous operation from the beginning. It seems to the writer that although this may be true, yet therein lies the real difficulty. It is a practical impossibility to attain this ideal condition of affairs. The beginning is past and gone, and the future is uncertain. Many public utility corporations have been content in the past, or in the early part of their development period, with revenues that were far from adequate, simply because of their hope and their faith that the future would enable them to operate on revenues sufficiently remunerative to compensate for the present; but the usual situation is that when the future arrives and their hopes are beginning to be realized, there comes the agitation for rate regulation and reduction in the revenues. The fact is that the real question at issue is to determine the starting point, and what is a fair revenue for the particular property under consideration at the present time and in the future. Surely there must be some means to determine what is a fair revenue for a property without reference to its past history, for, in a large majority of public service properties, there is no reliable past history from which sufficient data can be secured to serve as a basis of equity in the future. Furthermore, what may have transpired in the past cannot in equity be taken into consideration in fixing rates for the future, because it is evident that the sole purpose of rate regulation is to secure justice between the public utility and its patrons. The public in many cases has not been satisfied, either with the service or the charges made heretofore, and it is to secure equity in the future that the necessity arises for investigating public utility corporations. It is apparent that no past inequalities can be taken into consideration in fixing a rate for the future, because, in many cases, the present owners of the public utilities have not been the owners during the time when the inequalities may have been the greatest. Furthermore, the public of to-day and of the future are not the same individuals with whom the corporation has had its past dealings. Therefore, the fact that the rates may have been insufficient in the past, either in the early history of the corporation or throughout its entire life, cannot be considered

as justification for fixing future rates on a higher plane than that of equity between the corporation and the rate-payers, as determined by the present value of the property; neither can the fact that the rates in the past may have been more than adequate be considered at the present time as a justification for reducing them for the future below this same plane of equity, based on the present valuation. It is evident, therefore, that to introduce the factor of liberality in fixing rates for the future to compensate for past insufficiencies in return, or *vice versa*, would be an exceedingly dangerous precedent for a commission to establish in striving for equity in the solution of public utility problems; or, in other words, the law is neither for charity nor for vengeance, but for equity. Mr. Burns

In conclusion, the writer submits that the prime difficulty in fixing any system of rates is to secure an equitable starting point; but, as a general rule, the nearest approach to fairness and equity to all interests may be secured theoretically by ignoring the past history of the property and taking a new start, using the undepreciated cost of reproduction as the basis of all computations and the full life terms of each item of the property in the computation of the replacement fund, on the sinking-fund basis, and safeguarding the public by proper means which will guarantee that the utility will maintain the efficiency of the property consistently with the unlimited-life theory. In many cases, however, this theoretical condition cannot be attained owing to State laws that impose certain limitations upon the treatment of the question, or, owing to the interpretation of the law in accordance with the individual notions of certain commissions, and many other influences which the appraiser must recognize in the practical presentation of his problem. It is not meant by this statement that an appraiser should allow any State commission to deflect his judgment from what he believes to be the right results, but rather to convey the idea that it may be futile to present a subject in direct opposition to the rules and regulations of the commission, even though those rules are improper and unwarranted.

In many cases it is more practical, therefore, to compute the depreciation of the property and use the depreciated value as the basis of rates, keeping in mind the fact that the average public utility cannot attain a depreciation of more than approximately 30 or 35%, and seldom does exceed one of from 10 to 15 per cent.

Take, for example, a water-works system, the average expectancy of which will ordinarily be, say, 60 years. The maximum depreciation that it can attain and remain in operating condition is reached after it passes the age of complete renewal of all its parts and is living its second life. When it reaches this state, it has an average age of 30 years, an expectancy of 30 years, and, with liquid capital at 3% interest, the entire property has a depreciation of 29 per cent. This

Mr. Burns.

may be termed the ultimate limit of depreciation of this property, and as this ultimate limit can never be reached in any growing property, and usually seldom exceeds 15% of the cost of reproduction, it is not vital whether the reproduction cost of the depreciated value be used, in view of the fact that the replacement allowance must be greater in the latter case than in the former.

In reviewing a list of appraisals of public utilities valued by the writer, comprising water-works, electric plants, gas works, and telephone systems, aggregating a reproduction cost of $21 329 494, it is found that the aggregate depreciation for this entire list is $2 833 441, or an average of approximately 13%, and in using the depreciated value in each of these properties, it is found that but little difference is reflected into the rates, as compared with what would have resulted from the perhaps more theoretically correct method of using undepreciated reproduction cost.

Mr. Gillette.

HALBERT P. GILLETTE,* M. AM. SOC. C. E. (by letter).—Depreciation is loss of value. It is not loss of cost. There is depreciated value, but there is no such thing as depreciated cost. Accrued depreciation, therefore, must be associated with value if it is to have any real significance.

The value of a productive property is the present worth of its prospective net earnings, or, as commonly expressed, its "capitalized net earnings". In reference to plants, the term "value" can be used in no other sense and still retain its commonly accepted meaning.

Any measure of accrued depreciation must necessarily be a measure of value. The only criterion by which we can measure correctly the value of an old plant unit is a new plant unit capable of giving the same service in the most economic manner.

We have this general rule for determining the value of an old plant unit: Assuming gross income to be constant, the depreciated value of an old plant unit is equal to the first cost of a new plant unit of most economic design minus the capitalized difference in their equated annual operating expenses during the prospective economic life.

Operating expenses in this rule must include repairs and taxes, but must exclude functional, that is, commercial, depreciation and interest charges. The interest rate used in "capitalizing" must include both the normal rate of return on capital and a percentage sufficient to provide sinking-fund annuities that will retire the plant unit at the end of its prospective functional life. The annual operating expenses must be equated thus: To equate operating expenses, calculate the total cost of the operating expense of each year at compound interest up to the end of the last year of economic life of the plant unit.

* Chicago, Ill.

Mr. Gillette.

Add these compounded costs together and multiply by the annual deposit in a sinking fund which will redeem $1 at the end of the economic life. The product is the equated annual operating expense.

When the rule for depreciated value, previously given, yields a value equal to or less than the salvage value of any plant unit, that plant unit should be retired, because it is no longer economic. The rule, in practice, must be applied to the various units or groups of similar units of equal age that constitute the plant. There is no other rule or method that yields a theoretically correct depreciated value.

The depreciated value of a plant, derived by this rule, or by any other rule that approximates its results, must never be taken alone as a base for rate-making, for this depreciated value of the plant is only part of the total value of the property. The total value of the property, of which the plant is a part, is the present worth of its prospective net earnings. Hence, depreciated value is only part of the total value of which the other part is capitalized net profits. Thus, the total property value is $v = a + b$.

In this equation, a is the depreciated value of the plant and b is the remaining value of the property, or the non-physical value. If b is eliminated from consideration, then a has no useful significance in a rate case. Perhaps this fact is best illustrated by the value of city real estate, that is, land plus buildings. The value of the land is the capitalized net ground rent, which is deduced by subtracting the depreciated value of the building from the capitalized prospective net rent of the entire property. Eliminate the ground rent from consideration and the entire valuation problem vanishes, for the value of the building depends on its association with valuable ground. Similarly, as to a plant and the business that comes with its use, if the business is eliminated, the plant ceases to have more than salvage value.

In the sophistical reasoning so common in rate decisions, it is held that $v = a +$, etc. Then the "etc." is either forgotten or is arrived at by a cost method, for example, development cost by the deficit method. This is not reasoning; it is, at best, compromising; but, it is urged, commercial value cannot be a basis for rates, for that involves circular reasoning. If this is granted, then it follows logically that no commercial value of any kind can enter the rate-making problem. And as depreciated value is the commercial value of a plant, depreciated value, according to this hypothesis, can no more be used in a rate-making problem than can any other sort of value.

The units of an old plant unquestionably have less value than if they were new; but, conversely, the non-physical element has a greater value in an old property than in a new one, other conditions being equal.

Mr. Gillette.

The fundamental injustice in the ordinary rate-case decision lies in recognizing the loss of value of plant occurring with age, while refusing to recognize the co-existent increment in non-physical value also occurring with age.

Depreciated value must always be associated with capitalized net profits if there is to be a logical interpretation of the value of a productive property.

From another point of view, it may be seen that accrued depreciation cannot be logically considered apart from prospective net earnings. Let a commission enforce the rule that depreciated value is a base for rate-making. It would then follow that no astute public service company would undertake to improve its own plant by invention of new plant units, for to invent a new unit of a given class would result in depreciating all the existing units of the same class. An appraisal just prior to the replacement of the old units by the newly invented units would give the former only scrap value in such a case. An appraisal immediately following the replacement of old by new units would give a return on the undepreciated value of the new units, it is true, but would exclude all the old units that had been scrapped. In either case, the company would lose the amount of the accrued depreciation of the old units, and the loss would be consequent on the company's own efforts to reduce operating costs. Thus we reduce to an absurdity the proposition that rates should be based on depreciated value where depreciation is the result of invention.

The same sort of reasoning applies to depreciation due to economic inadequacy. Instead of putting in a small number of large plant units and replacing a larger number of small units, a utility or railway company would find it more profitable to avoid the functional depreciation thus caused, and it would add more small units to its plant. The result would be an uneconomic plant, viewed as we now view economy; but it would be economic to the company, viewed as it is proposed to view depreciated value as a basis for rates. Under such conditions, a public service company or railway would be foolish to encourage invention or other action to change existing plant units. Not only would it discourage its own engineers and other employees, but it would frown on the efforts of outsiders to effect plant improvements. If forced by a commission to install a new plant unit, the company would endeavor to keep all its old plant units in "use", if only as standby equipment, so as to secure a return on the old investment.

This *reductio ad absurdum* serves to illustrate clearly the fallacy of basing rates on depreciated value, for it is apparent that depreciated value of plant units has no true economic significance save in connection with the value of the net earnings of the entire property. Rates might be based on actual cost of plant, and still yield an

economic absurdity, but to base rates on depreciated value is to pile the absurdity even higher. Because of the distinctions between cost and value, between the agency theory and the competitive theory, and between present and past standards of equity, it is difficult to avoid logical pitfalls in a rate case. Many engineers are badly confused as to appraisal principles, so it is not to be wondered that commissions and Courts still err, not only as to depreciation, but as to many other matters. Let us, however, not deny the existence of accrued depreciation; but let us point out that depreciated value cannot logically, or fairly, or economically, be taken as a base on which to calculate the total "fair return" to investors in railways and public utilities. Mr. Gillette.

W. KIERSTED,* M. AM. SOC. C. E. (by letter).—The author of this paper has taken pains to define carefully the various terms used in his very interesting discussion of the question of depreciation as an element to be considered in the valuation of public utility properties; but, notwithstanding all the care thus bestowed on the definition of terms, it is a question whether the atmosphere is altogether cleared of confusion. Mr. Kiersted.

For instance, in the definition of "fair value", the author states:

"The term 'fair value' as used in connection with rate-fixing is difficult to define. * * *

"The value to an investor is unhesitatingly determined from the net earnings, with due regard to hazards of the business. The value for rate-fixing purposes is to be that value on which, with the same regard for the hazards of the business, the owner is to be allowed to earn a fair interest return. Value should be the same whether determined by a rate-fixing body or whether determined for a purchaser. 'Fair value', then, is necessarily based on proper and reasonable investment which may be ascertained from cost records, and, when cost records are not available, is usually estimated by the 'cost-of-reproduction' method."

Confining the remarks of the writer to public utilities of a monopolistic character, like municipal water-works, lighting plants, and other properties of a similar character, it is clear at the outstart that, as the author states, value to an investor in any of these public utilities is determined from the net earnings. It is not so clear that net earnings should invariably provide a fixed rate of return on money invested in property actually in use, for this depends on social and industrial conditions, whether retrograding, stable, or improving. In any event, value, being altogether dependent on earning capacity, is an element of consideration only after a public service commission shall have concluded its labor and definitely fixed a rate of return for the property. To regard value as an element of consideration

* Kansas City, Mo.

Mr. Kiersted.

before the conclusion of a commission's work is putting the cart before the horse. Investment and value are not synonymous terms.

The work of a commission possesses really two independent functions: one is the determination of a basis for rate-making; the other is the process of rate-making. Value is the result of this twofold operation. If the public service commission fixes a rate of return below that which will invite investments in a particular kind of public utility, capital is deterred from investment and value is depreciated; on the other hand, if the rate of return is equal to or above an inviting rate, no trouble is likely to arise in securing capital for the improvement and extension of the property as needed, and value is either stable or enhanced.

The point which the writer desires to make is that ordinarily the work of the engineer in a rate case is not that of determining value, but rather a work of determining a basis for rate-making on which a rate of return may be predicated. The actual determination of value is the work of the public service commission in fixing the rate of return. The commission actually makes value. If the service rates are modified by a commission in a manner to change net earnings, the value of the property becomes changed proportionately. Value is a relative, not a definite, term, becoming in academic discussions a sort of "will-o'-the-wisp". Therefore, if engineers in the accomplishment of their work would leave out of consideration entirely the idea of "value", the confusion which arises in attempting to reason vaguely and academically from cost to value or from investment to value would largely disappear. The writer, therefore, would suggest that valuation work in a rate case be regarded as an effort to arrive at a basis for rate-making, or, to put the matter more simply, to determine a rate-making base as a foundation for the work of a public service commission in fixing a rate of return. The engineer's work may be carried even a little farther than this, in an endeavor to throw light on what constitutes a proper replacement return to maintain the particular property which may be under consideration at the time at a 100% value. The other portion of net earnings, the interest return on the invested capital, is largely a matter of business, and is not to be determined purely as an engineering problem.

The principles which the author outlines as peculiar to the "Unlimited-Life" method of valuation are not new. They have been recognized on several occasions. The writer applied them in his testimony on the valuation of the water-works property of Los Angeles, Cal., in January, 1899, when the property was being appraised by a commission for purchase by the city.

In this case the writer ascertained the commercial value of the property by capitalizing the net earnings, and, in his discussion of

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the reasons for pursuing this method, stated, among other things, that: "the income of a water-works property, after deducting the actual operating expenses, should afford a net income (earning) which should pay a proper interest upon the investment in plant and property establishment and in addition thereto should support a general maintenance fund for expenditures in betterments which will effectually perpetuate the life of the property. * * * It should be stated in this connection that the application of the maintenance fund should not be to displace alone the materials of construction which have wasted away through the agency of natural decay, but also to reinforce or displace those portions of the plant which may have been incapacitated by reason of plant expansion in response to a demand for an additional supply of water, or which may have become inadequate by reason of modern improvements, as is often the fate of machinery. * * * This method of valuation implies an indefinite continuance of earnings. But this inevitably must be the condition of a property which is a part and parcel of the established property of a progressive city, regardless of the question of vested title or the existence or non-existence of a franchise. For whenever the franchise rights shall cease by limitation and the water company operating under it no longer exists legally, still the property remains a living business, because a continuous operation of it cannot cease without injury to private and public interests. * * * Upon a transfer of possession after due compensation, the property remains intact as before the transfer, the net earnings remain an established fact, and continue to remain indefinitely susceptible to apportionment for the support of the invesment and for the maintenance of the perpetuating fund. Therefore it is maintained that valuation by the capitalization of net earnings based upon equitable rates, when proper apportionment is made to perpetuate the plant, becomes proper. But the valuation of a property determined in this manner embraces a plant which is capable of rendering adequate service of the kind contracted for. Consequently, if proper expenditures have not been made progressively to maintain the efficiency of the plant, then at the time of appraisement such deduction should be made, consistently, from the capitalized commercial value of the property as a whole, as will represent the necessary expenditures to produce a water-works plant of adequate capacity. In making this deduction the question of physical deterioration is subordinate to that of the useful life of the materials of construction. Though it is conceded that physical deterioration is in progress continually, still the rate, range, and extent of such deterioration cannot be definitely estimated, to say nothing of placing thereon a tangible value, nor can the limit of useful life be predicated upon such a basis. On the other hand, it must be conceded that the continuance, even of an inferior service, demands the exercise of due vigilance on the part of the management of the property to replace useless material, for, in a united system, no matter how complex it may be, a temporary disablement of any essential part of a system impairs the efficiency of the whole to a greater or less degree. Expenditures from the maintenance fund have removed such defects as may have been discovered in the past, and they undoubtedly can be made to eliminate them in

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the future. When the cost of (neglected) reinforcements and renewals to prolong or renew the useful life of the plant are estimated and deductions from a commercial valuation are made accordingly, then all has been done that can be done, consistently, in estimating the depreciating effect of deterioration."

The same principles, with regard to depreciation of the commercial value of a water-works property, is referred to more comprehensively in the writer's discussion* of his paper "Valuation of Water-Works Property" presented in April, 1897, in which the following statement is made:

"Should certain portions of the plant, however, be inefficient, the necessary cost to reinforce the deficient portions should be deducted from the commercial valuation derived from the net earnings; or, if reinforcement be impracticable, to replace it with such new portions of plant as will fulfill the requirements of an efficient service. * * *

"A rule of this kind should be comprehensive; therefore, if a water company has distinctly built for the future, then to the commercial valuation should be added the cost of the parts which safely and distinctly provide for this future; * * *."

Although there can be no question that the way of ascertaining the value to an investor of an unregulated public utility is by the capitalization of earnings, with due regard perhaps to the future, as well as to the particulars of deferred maintenance, under-construction, and over-construction at the time of valuation, as set forth in the foregoing quotations, still the Courts seem to have considered this method of valuation as not altogether conclusive. This method of ascertaining value may have suffered like others in its presentation; but, however this may be, such a method can have no significance in a rate case until the work of rate-making in all its particulars shall have been completed. Whenever the net earnings resulting from the rate-making processes become known, then, and only then, does it become a factor in ascertaining value.

In the ascertainment of a basis for rate-making, or of a rate-making base, the writer agrees with the author that there is much in the investment, unlimited-life, no-depreciation method deserving thorough consideration. Doubtless many of those interested in valuation work for rate-making, as are the engineers, and of those engaged in the work of rate-making itself, as are the public service commissioners, have given the matter consideration; but they are far from united in their views. The logic of those attempting to frame methods of valuation to suit the decision of the higher Courts is not free from serious criticism. Others, willing to accept the investment and unlimited-life theory, hesitate at the no-depreciation part of the theory, ostensibly on the ground that the Courts have not as yet approved of allowing

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, pp. 208–209.

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no deductions for depreciation. The reason for this view of the Court may be due largely to the failure of appraisers in the past to confine themselves strictly to the investment theory, as well as to the manner in which these matters have been presented to the Court.

It is necessary, therefore, that the investment, unlimited-life, no-depreciation method be defined clearly if the hope is entertained of its acceptance by the Court. In this connection it is not clear that the term "investment" has been defined explicitly. If the view of a California banker, once stated to the writer, "once an investment always an investment", is to be accepted in its broadest sense, there may be grave doubts of the investment theory ever receiving recognition. It is necessary, therefore, to define investment. The author defines "investment or capital invested" as

"the aggregate of the reasonable and proper expenditures which have been made to render the property in question efficient for the purpose for which it is intended."

Investment is further referred to as follows:

"In regulating rates, consideration need be given only to the amount of capital reasonably and properly invested, without any deduction for depreciation.

* * * * * * * * * *

"It is to be expected, therefore, that in the case of many public service properties the owner will have had no opportunity to make investments the earnings of which will meet his replacement requirements. When there has been such a deficiency of earnings, the ordinary method of computing the annual replacement increment by sinking-fund, compound-interest methods, based on the original probable life or on the reduced value computed by any method and the remaining life, would be improper and inadequate. It is under such circumstances that recourse must be had to special methods of computing the depreciation or the annual replacement increment.

"In such cases as those just referred to it may be necessary to provide for a replacement at the full cost of various parts in the remaining years of life, or burdening the property with an allowance for the accrued deficiency of earnings, which is then to be amortized in a reasonable number of years.

* * * * * * * * * *

"Under the Unlimited-Life Method there has been no return of capital. The appraisal is at 100% of the investment.

* * * * * * * * * *

"When, however, the appraisal is made on the assumption of unlimited life and the owner is assured a fair interest return on the full amount of his investment besides replacement allowance as the same becomes necessary, the property will have a 100% bonding value, * * *."

The question arising is, what is meant by "investment" or "capital invested" in a specific case; for instance, a water-works property which has been carefully inventoried. The inventory is made to include

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simply the various items of property in use and useful to the operating utility, and to exclude all reference to the items of property which have been abandoned or replaced, for one cause or another, but which may be still carried on the books as a part of the capital investment. Is the author's view of the investment for rate-making purposes confined to the inventoried property, or does it embrace all the property carried in the capital account? It is presumed, if it refers only to the inventoried property, that the owner must have been compensated through a replacement fund for all the property which is abandoned or for any reason displaced; but, in the event that the earnings have not been sufficient in the past to amortize the original cost of the abandoned articles, it is presumed the author considers that the investment in such items of property should be carried in the capital account until amortized. Thus, the capital account would be composed of two distinct elements, one of unlimited life perpetuated from replacement earnings, the other of limited-life in process of amortization. As an alternative, assume that unlimited life applies to both elements in the capital account, without any provision for amortization; then the owner would be drawing interest continually on the abandoned property, and interest and replacement on living property. On the other hand, if all abandoned or displaced property is charged off the capital account, as it should be at the time of replacement, regardless of whether the owner has or has not been reimbursed, and in its place additions are made to the investment in the property inventoried in an amount representing the past deficits, then the situation is again complicated under the unlimited-life theory because of the fact that the deficits should remain in the capital account only until amortized. Again, there is the combination of limited-life and unlimited-life elements in the investment.

These inconsistencies may be eliminated by restricting the rate-making base to the investment in inventoried property with such additions for interest, contingencies, development cost, etc., as may be fair and proper, allowed once for all, leaving out of consideration altogether all questions relating to the amortization of the original cost of abandoned or displaced property, and considering past deficits to such extent only as may be open for consideration as a part of operating expense to be carried in the operating account until amortized in some limited period of time that may suit a particular case. This would apply to property that is being appraised for the first time for rate-making purposes by a public service commission. Thereafter replaced items of property should be charged off capital account, as replacements may be made, and the substitute article added thereto, with due allowance for scrap value—the rates being modified by the public service commission from time to time as occasion may require, but always predicated on the rate-making base, as at first determined,

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corrected by additions and subtractions in the manner above indicated, with due allowance for interest and other expense properly chargeable to piecemeal construction.

Whether deductions for depreciation shall or shall not be made in determining a rate-making base depends on the method pursued in making the appraisal. It can scarcely be conceived that a public service commission should confine itself to any particular method of valuation. Undoubtedly, a careful audit should be made of construction accounts with a view of determining as far as possible the cost of the property in use at the time of the appraisal. In a similar manner, an audit should be made of income, disbursements, and earnings. However, such an audit, under most circumstances, can scarcely be conclusive evidence of actual investment.

The records of the cost of construction of most of the smaller public utilities, and perhaps of some of the larger ones, can be verified by estimating the cost of construction made piecemeal by the use of unit prices made up to suit the price of materials and labor prevailing at the time each particular piece of work was done. This procedure, of course, necessitates a careful search of the construction records, and the tabulation of the construction work in a manner to show the original construction and the annual construction increments from year to year. In pursuing this method, considerable benefit is to be derived from a tabulation of the prices of materials and labor showing the variations from year to year. Doubtless a search will disclose the existence of much desirable information of the kind. A late investigation by the writer along this line is illustrated by two typical labor diagrams, Figs. 8 and 9. Fig. 8 represents the general percentage variation of the hourly wages paid to bricklayers from 1840 to 1913, and Fig. 9 represents the hourly wages paid to laborers for the same period. Each diagram presents a double curve during the Reconstruction period, after the Civil War; one curve, through a series of points (circles), shows the percentage variation of wages paid in currency, the other curve shows the equivalent variation for payments in gold. A similar set of diagrams for each class of labor and for manufactured materials, like cement, cast-iron water pipe, copper wire, machinery, and similar products subject to market fluctuations, similarly developed, would enable one to estimate with reasonable accuracy the probable investment in a property constructed piecemeal. In determining labor costs applying to the various dates of piecemeal construction for a particular locality, it will probably be found possible to trace out smoother and more regular curves than those shown on these diagrams.

In the absence of records showing the piecemeal development of a property, the reproduction method of estimating cost would have to be pursued; but the use of that method should entail a modification

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of some, at least, of the present processes, if no deduction is to be made for depreciation in arriving at a rate-making base, as, for instance, a modification to exclude undue weight for those elements frequently referred to as the unearned increment, some of which, like the paving over pipe lines placed after the pipe lines themselves were constructed, have aroused strenuous objection. In this connection the author

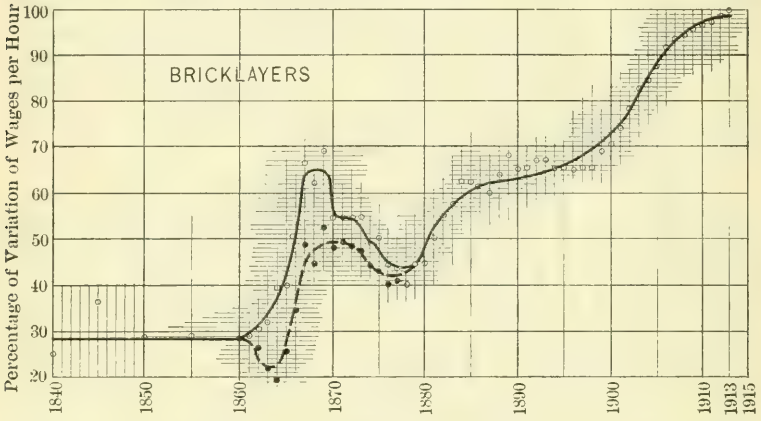

FIG. 8.

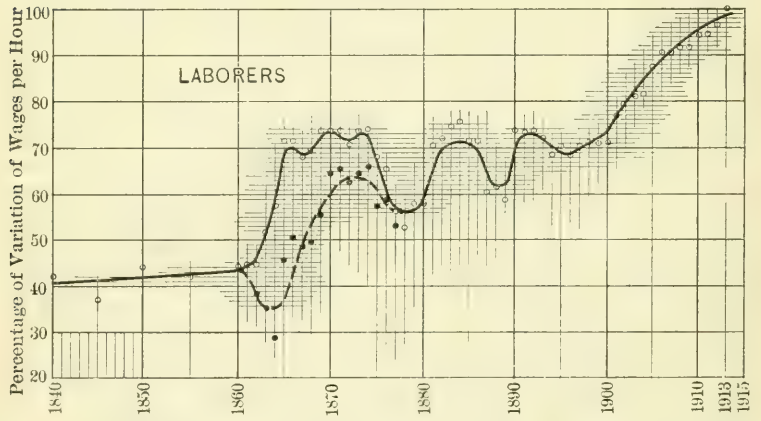

FIG. 9.

allows the present value of real estate by regarding the increased value thereof as reinvested earnings. Still another reason, referred to by the writer in several previous discussions, is that the present value of real estate should be allowed, on the presumption that the increase in value is due to urban surroundings, particularly in a growing city, rather than to its identification with a particular public utility. In

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other words, present value of real estate should be considered on the ground of its usefulness for purposes aside from that due to its particular association with a public utility, entitling it to different consideration than that accorded items of property possessing value that is due wholly to a specific use as part of an operating property.

Before passing this part of the discussion it may be pertinent to add a word with reference to an estimate of reproduction cost under original conditions at present-day prices of materials and labor and under present-day methods of construction. It is a compromise of the investment method suggested by the author and the method of reproduction under present-day conditions as ordinarily pursued, with a view undoubtedly of avoiding obvious difficulties peculiar in both methods, respectively, when logically pursued. It would seem, however, that the historical records and the book accounts affording explicit information as to past physical conditions would present with equal completeness the actual cost under the original conditions and would otherwise be sufficiently explicit as to dates, etc., to enable one to apply the prices of material and labor prevailing at the time of construction with sufficient accuracy to test the book account of costs.

With judicious discrimination of the kind indicated in estimating the probable piecemeal investment in a public utility, or the probable cost of reproduction as estimated by a restricted method, together with a comparison with actual cost records contained in the book accounts, and with due allowance for development expense, interest, and the necessary overheads, it would seem, with regard to many of the monopolistic utilities, that there should be no serious difficulty in arriving at a rate-making base without further allowance for depreciation than that introduced in a process of valuation which excludes consideration of investments made in property abandoned or which, for one cause or another, may have passed out of use. The determination of a rate-making base is regarded by the writer as separate and distinct from the determination of the value of a property for purchase and sale.

Before concluding this discussion the writer desires to call attention to the importance of selecting proper unit prices in valuations for a rate-making base. Each unit price should be thoroughly investigated before use in valuation work, and the various steps taken in the make-up of the unit price for each important item of property should be presented analytically. Such procedure will tend to avoid the unbalanced estimates of physical property sometimes encountered in valuation proceedings, and will compel the engineer making the estimates to be thorough in his work and careful in his selections. With few exceptions, the more nearly this analysis conforms in its make-up with past physical conditions and past market prices of labor

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and material, the more nearly will the ends of justice be served in determining the amount of money actually and fairly invested.

In conclusion, the writer would summarize the points he has endeavored to bring out in this discussion, with due regard to the stated restrictions, as follows:

1.—The process of rate-making consists of two independent operations: one is the determination of a rate-making base, the other is the determination of a schedule of rates which will insure proper return on an investment previously ascertained.

2.—The value of a public utility rendering an efficient service is a function of net earnings.

3.—Valuation is the initial step in rate-making, and, so far as it goes and can go, is not in itself a method of determining value. Accordingly, all idea of value should be excluded from the process of valuation for rate-making. Valuation in a rate case should be directed to the determination of a rate-making base.

4.—Investment in property in use and useful for the rendering of an efficient service and in the judicious development of such property is a term synonymous with rate-making base.

5.—Investment thus defined may be determined in part from construction accounts, or from estimates of the cost of constructing and developing the property piecemeal, or from estimates of reproducing the property under substantially present-day conditions, when excluding undue consideration of the unearned increment. As a rule, the determination of a rate-making base should give due consideration to all three methods herein referred to.

6.—From a rate-making base determined in the manner above stated no deductions should be made for depreciation.

7.—Investments in public utilities, as a rule, should be regarded as of unlimited or perpetual life, and in addition to earning a fair rate of return, they should earn sufficient to replace items of property as they become worn out or obsolete, and to provide reasonably for unforeseen contingencies. The obligation to replace such items of property is direct and imperative. Replacements thus made perpetuate the life of the property.

8.—Elements of cost open to consideration, but which should not remain permanently a part of the rate-making base, should be amortized as a part of operating expense during such term of years as it may be desirable to fix in any particular instance.

9.—Properties under regulation by public service commissions should have added to the rate-making base the net accretions made to the investment from time to time as the property is extended and improved.

10.—Unit prices used in making up a rate-making base should be carefully selected, compiled, and itemized.

11.—The rate of return on the rate-making base is not always controlled by the hazards to which an investment is exposed, nor by the rate of return common to other similar investments, for the reason that social and industrial conditions must receive due consideration. That is to say, declining social and industrial conditions in a town or district must result in a decline of earnings and in a small rate of return on a fixed rate-making base. Mr. Kiersted.

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PAPERS AND DISCUSSIONS

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THE WATER-PROOFING OF SOLID STEEL-FLOOR RAILROAD BRIDGES

Discussion.*

By Messrs. A. W. Carpenter, C. T. DeLamere, and John Jervis Vail.

Mr. Carpenter.

A. W. Carpenter,† M. Am. Soc. C. E.—The speaker has been very much interested in this paper, as he has had to design and write specifications for water-proofing for many similar structures. The lines he has followed have been somewhat similar to those set forth by Mr. Wagner. In a good many respects, however, they have been different, and perhaps some of his experiences and differences of practice may be of interest.

The art of water-proofing solid steel-floor railroad bridges is, as has been intimated, a rather new one, and in the process of development. One of the discussions mentioned the earliest form, at least the earliest known to the speaker, which is the so-called binder that was put in the bottoms of the steel troughs which formed the earliest type of the solid floor.

That binder, although entirely inadequate as a water-proofing material, had the virtue of affording very considerable protection to the steelwork. In a typical case the lower inside surfaces of the troughs were protected only by a deposit of this so-called binder, which was composed of either coal-tar pitch or asphalt, mixed with gravel or sand; or, in some cases, it was thought fit to specify a mixture of coal-tar pitch and asphalt, and then mix that with sand or gravel. A great many of these old troughs, built from 1888 on, which were uncovered many years later, showed that the binder had protected the steel from corrosion very well.

* Discussion of the paper by Samuel Tobias Wagner, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† New York City.

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The more elaborate form of water-proofing, which is the subject of this discussion, appears to have been introduced about the beginning of this century, and has been, of course, largely developed in connection with the extension of grade-crossing elimination work in cities.

Mr. Wagner mentions ballast as being one of the main reasons for requiring water-proofing. The speaker thinks that is so, and that one of the ways to get rid of the necessity for water-proofing is to get rid of the ballast. He has experimented to some extent on solid floors with the ties or timber rail blocks embedded in concrete in place of ballast. That does away with the necessity for elaborate water-proofing. A smooth surface of concrete properly sloped carries the water off rapidly, so that it can be taken care of by gutters.

Mr. Wagner thinks it inadvisable to attempt to drain the water over the bridge ends and back walls. The speaker has always thought this the preferable way, where the bridge is on a sufficient grade to obtain such drainage inexpensively, as it generally dispenses with the necessity for the expensive and frequently troublesome drainage system of outlets, gutters, and down-spouts. A grade of 0.7 of 1% on short bridges has been considered sufficient for this purpose.

With regard to carrying water-proofing and concrete over the top of low half-through girders between tracks, the speaker would not advocate this if any increase of track spacing in order to maintain proper clearance should be caused thereby, thinking it would not be worth the cost.

Mr. Wagner objects to girder flashing angles. The speaker has found that form of construction very satisfactory. Instead of stopping at the stiffeners, he has carried the steel flashing around the stiffeners and knee-braces. He realizes that the bridge shop does not like it very well. It involves a great deal of difficult detail, but it seems to give the desired result, so far as the water-proofing is concerned.

Mr. Wagner refers to only one water-proofing material, that is, asphalt. The railroad with which the speaker is connected has used both asphalt and coal-tar pitch, and has had good results with each. Coal-tar pitch is cheaper, having the advantage in that respect. With regard to durability, as far as the speaker's knowledge goes, coal-tar pitch is very durable. He is not so sure about asphalt.

Some fourteen bridges, carrying railroad tracks over streets in Buffalo, and designed under the speaker's direction, are water-proofed variously with coal-tar pitch and with asphaltic compounds, largely according to the methods described by Mr. Wagner. They have been in service from 4 to 6 years, and are all reported to be satisfactory, as far as water-tightness and the general condition of the water-proofing are concerned.

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With reference to the use of burlap in the water-proofing membrane, the speaker thoroughly agrees with Mr. Wagner's statement in Appendix B, that it should never be used in the untreated or raw state, and he has doubts as to the possibility of overcoming its bad properties by treatment. Treated cotton fabric seems to be preferable.

Referring to Mr. Wagner's specifications for water-proofing, the speaker offers the following comments:

Paragraph 1.—"Depth", in this paragraph, refers to the depth of steel or concrete construction. It would seem rather more logical, in a specification for water-proofing, to refer to the depth required for the water-proofing itself. In the speaker's practice it has been found that one can work with a minimum of 2 in. for the water-proofing itself plus the protective covering, using $1\frac{1}{2}$ in. of reinforced cement mortar for the protective coating; and with a steel-plate floor, 4 to $4\frac{1}{2}$ in. from the top of the plate to the top of the protection, 4 in. with cement mortar protection, and $4\frac{1}{2}$ in. with brick.

Paragraph 2.—For providing a drainage slope, the speaker has successfully used reinforced cement mortar with a minimum thickness of $1\frac{1}{2}$ in.

Paragraph 3.—The speaker has thought the closed form of drains specified (and illustrated) by Mr. Wagner rather objectionable for bridges in cold climates, and for that reason he has used inlets leading to open gutters.

One point that Mr. Wagner illustrates but does not mention in his specification, is that it is not a good thing to put the drains or inlets directly under the tracks. A number of the earlier bridges had that construction, and it was difficult to get at the drains to clean them.

Paragraph 4.—This paragraph provides, in effect, for continuous girder construction, which in some cases would be unsatisfactory.

Paragraph 5.—The necessity for this paragraph is not apparent, and in some cases the speaker would not want to use gusset-plates without angle reinforcement on the outer edges.

Paragraph 9.—The speaker would prefer to specify reinforced felt or treated cotton fabric in place of treated burlap.

Paragraph 10.—The thickness of reinforced cement protection may be reduced to $1\frac{1}{2}$ in. as a minimum.

Paragraph 12.—For insurance of absolute water-tightness, caulked edges of the steel at joints in the steelwork are to be recommended, although expensive. In addition to such caulking, certain openings would generally require caulking with lead, oakum, or other suitable material.

Paragraph 13.—Reinforced cement mortar, composed of 1 part cement and 3 parts of sand, minimum thickness $1\frac{1}{2}$ in., has been the speaker's standard for steel-plate decks.

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Paragraph 15.—Coal-tar paints would be preferable with coal-tar pitch water-proofing. The necessity for painting mortar surfaces, especially where coal-tar pitch is to be used, is not apparent.

Paragraph 18.—Concrete or mortar surfaces should be dry when paint or water-proofing is applied. Presumably, the treatment recommended is intended to insure dryness, but might not.

Paragraph 39.—The criss-cross form of laying water-proofing is novel to the speaker. The requirement that the first layer of water-proofing felt be not cemented to the bridge floor seems to be of doubtful utility; the opposite practice has obtained in all the speaker's bridges.

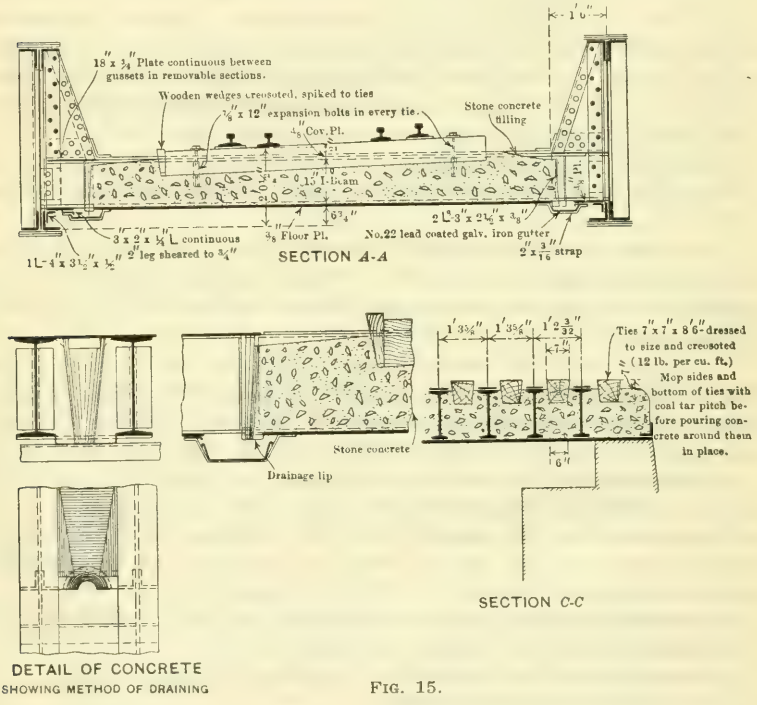
1'0"
18" x ¾" Plate continuous between gussets in removable sections.
Wooden wedges creosoted, spiked to ties
⅞" x 12" expansion bolts in every tie.
Stone concrete filling
⅝" Cov. Pl.
15" I-Beam
6¾"
3" x 2" x ¼" L continuous
⅜" Floor Pl.
No.22 lead coated galv. iron gutter
2" x 3/16" strap
1L-4" x 3½" x ½" 2" leg sheared to ¾"
SECTION A-A
1'3⅝"
1'3⅝"
1'2 3/32"
7"
Ties 7" x 7" x 8'6"-dressed to size and creosoted ≥ (12 lb. per cu. ft.)
Mop sides and bottom of ties with coal tar pitch before pouring concrete around them in place.
Stone concrete
Drainage lip
6"
SECTION C-C
DETAIL OF CONCRETE
SHOWING METHOD OF DRAINING

FIG. 15.

Some other points of criticism of the specifications are covered by the general discussion and in the specifications used in the speaker's work, herewith submitted.

Figs. 15 to 19 are reproductions of drawings for different types of water-proofing applied to bridge structures. Commenting on the latter:

Fig. 15 shows a floor with track ties set in concrete, and no other water-proofing. The openings along the girders leave the gutters ac-

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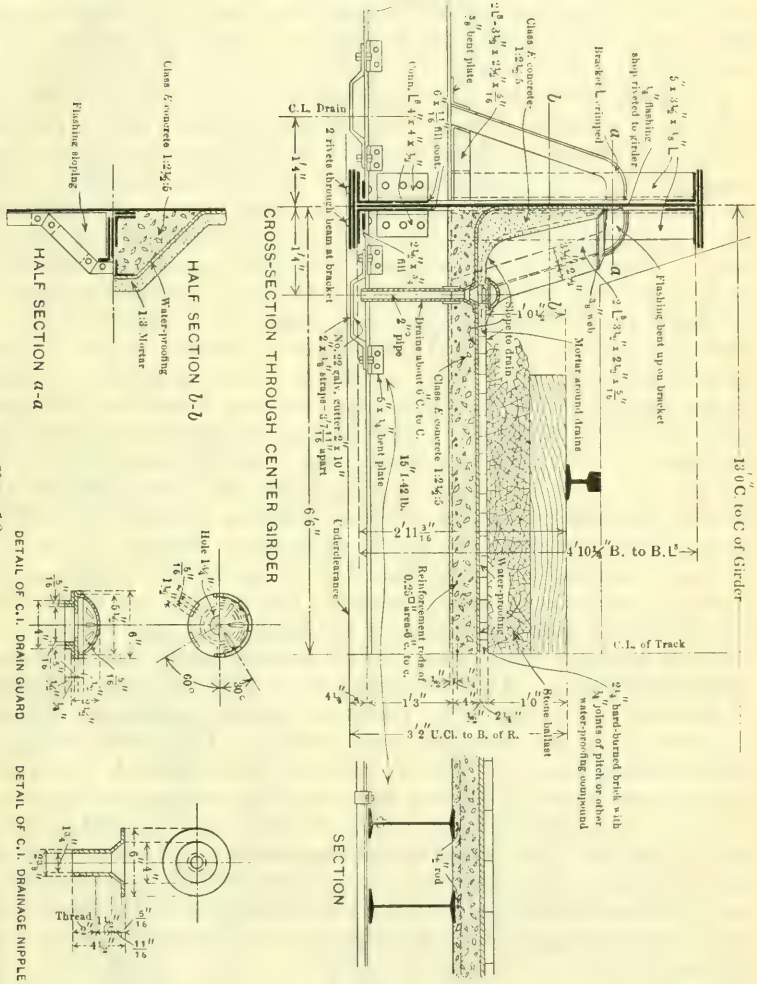

FIG. 16.

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cessible from above, and the floor connections open for inspection. They also cause some slight saving in load on the girders. The $\frac{1}{4}$-in. plate-covers prevent rubbish from falling into the openings and clogging the gutters.

Fig. 16 shows one-half of a single-track floor carried by through plate girders, with reinforced concrete floor-slab, fully water-proofed. The girders have flashing plates which are carried around the knee-braces.

The bridge for which this design was made carries four tracks with five lines of girders, and has a total length of about 114 ft. The cost of the water-proofing, which was done in 1910, was equivalent to 46 cents per sq. ft. of water-proofed area, this cost including the reinforced concrete floor-slab, the water-proofing proper, and the brick and concrete protection, but not including the girder flashing and the drains. The width of water-proofed area was taken as the full width of the floors plus the aggregate of the distances the water-proofing of the floors was extended up the girder webs. The floors were free of traffic when water-proofed. These details are mentioned because they are essential to a proper understanding of unit cost figures. Another bridge with a similar floor and water-proofing design and otherwise comparable, for which the water-proofing and protection only were contracted in 1911, cost for these items, the equivalent of 17 cents per sq. ft., based on similar measurement and under similar conditions. In both these cases the water-proofing material was coal-tar pitch. This figure shows the details of the drain castings.

Fig. 17 shows one double-track floor of a four-track, three-girder, through plate-girder bridge. The cross I-beam floor has a deck-plate with reinforced mortar covering of minimum thickness, furnishing a drainage surface, asphaltic-compound water-proofing, and brick and concrete protection. The deck-plates are turned up along the edges of the knee-braces, this being permitted by the considerable distance the girders are set from the track centers. The complications of water-proofing along the girder webs are thus obviated, the water-proofed area is greatly reduced and simplified, and the advantages of the open space along the girders are provided, as mentioned in reference to Fig. 15. This bridge crosses the intersection of two streets, so that its length varies, but it averages about 140 ft., and the cost of water-proofing in 1909 was equivalent to 21 cents per sq. ft., this including mortar drainage surface, water-proofing and protection, but not including drains.

Fig. 18 shows one single-track floor of a through plate-girder bridge with cross I-beam floor and concrete floor-slab. The floor is deep, and the girders do not extend far above the rails, so that the knee-braces have been concreted in to permit the water-proofing to be carried up on a plane parallel to and outside the outer edges of

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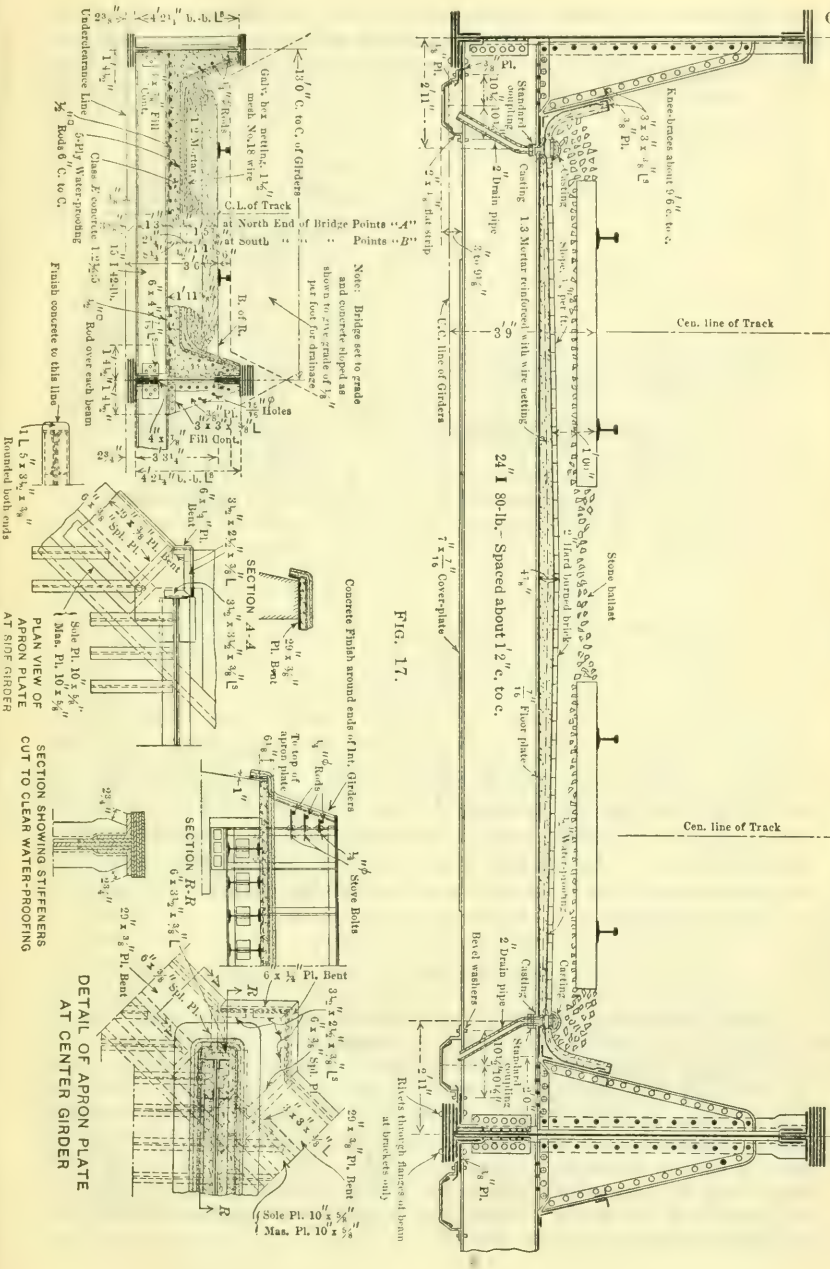
Knee-braces about 9'6" c. to c.
Cen. line of Track
24" I 80-lb.- Spaced about 1'2" c. to c.
Stone ballast
Floor plate
Cen. line of Track
Cen. line of Track
Rivets through flanges of beam at brackets only
Fill
Fill Cont.
Holes
Fill Cont.
Rounded both ends
Sole Pl. 10" x 5/8"
Mas. Pl. 10" x 5/8"
PLAN VIEW OF APRON PLATE AT SIDE GIRDER
Concrete Finish around ends of Int. Girders
Stove Bolts
SECTION R-R
SECTION SHOWING STIFFENERS CUT TO CLEAR WATER-PROOFING
DETAIL OF APRON PLATE AT CENTER GIRDER
Sole Pl. 10" x 5/8"
Mas. Pl. 10" x 5/8"

FIG. 17.

FIG. 18.

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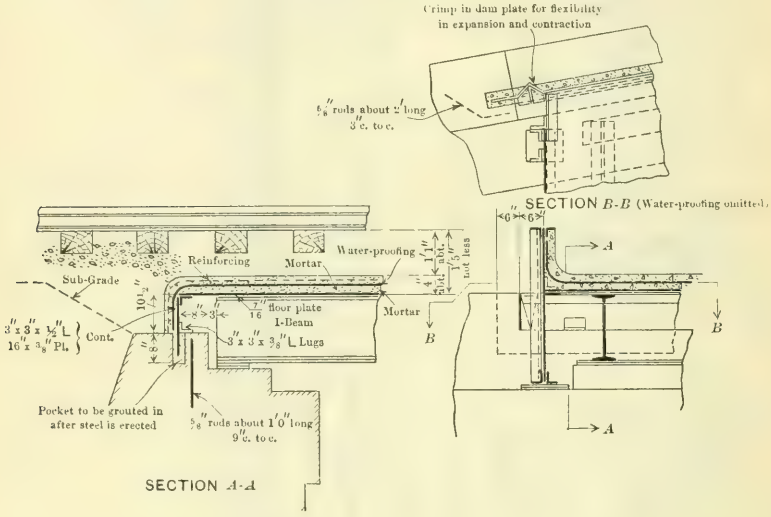
the braces to the under sides of the girder top flanges. The figure shows details of and finish of the steelwork and water-proofing. Apron-plates, bent over the back walls, are used in preference to any detail which would hold water on the steel deck in case of leakage through the water-proofing.



BACK WALL DETAIL
FOR DECK PLATE AND BEAM SPANS

FIG. 19.

Fig. 19 shows a design for the end finish of steelwork and water-proofing for deck-plate and I-beam bridges. This connects the steel superstructure with the masonry by a water-tight joint which is sufficiently flexible to permit longitudinal expansion and contraction of the superstructure. The design is not considered properly adapted to bridges with ends on considerable skews. It has been used successfully on several bridges.

SPECIFICATIONS FOR WATER-PROOFING BRIDGE FLOORS.

1.—Mortar Foundation for Bridges with Steel Deck Plates.

After thoroughly cleaning and drying the surface, apply to the metal one coat of coal-tar paint, as approved by the Engineer. On this coating spread a layer of cement mortar, composed of 1 part of Portland cement and 3 parts of sand, and reinforced with metal lath or wire netting. The mortar shall be placed in two layers of equal thickness, one following the other quickly, with the reinforcement between, so as to form a monolithic slab. At facia girders the slab shall be formed as shown on plans. The mortar slab is to be 1½ in. thick at drain nipples, with surfaces draining toward the nipples on a slope of ⅛ in. per ft.

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The mortar surface prepared to receive the water-proofing shall be smooth and true to grade. There shall be used four thicknesses of plain tarred felt, one thickness of reinforced felt, and a sufficient quantity of roofing pitch to provide for the moppings specified. These materials shall be used as follows:

II.—Water-Proofing.

First. Coat the entire surface of the concrete uniformly with pitch, in which, while hot, lay two plies of felt, lapping each over a width of the preceding one equal to one-half its total width plus 1 in. (this lap should be 17 in. in case the sheets are of the usual width of 32 in.), mopping with pitch the full width of the under sheet to be covered, so that in no place shall felt touch felt.

Second. Coat the entire surface uniformly with pitch, in which, while hot, lay one thickness of reinforced felt (uncoated side down), lapping each sheet 2 in. over the preceding one.

Third. Coat the entire surface uniformly with pitch, in which, while hot, lay two plies of felt, lapping the sheets as specified for the first operation.

Fourth. Coat the entire surface uniformly with pitch.

The felt water-proofing shall be turned down into the drainage nipples and at facia girders shall be turned up, as shown on plans, and stuck closely to the steelwork. The roofing pitch shall be heated to a temperature approved by the Engineer. Each coat shall cover completely and entirely the surface to which it is applied, without cracks or blow-holes, and the felt must be rolled into the pitch coating while the latter is hot, and must be pressed into it so as to insure the felt being completely stuck to the coating over the entire surface.

III.—Mortar Protection Course.

Over the water-proofing layer thus formed shall be laid a protection course of reinforced cement mortar, 1½ in. thick. The mortar shall be composed of 1 part of Portland cement and 3 parts of sand. The reinforcement shall consist of metal lath or wire netting. The mortar shall be placed in two simultaneous layers with the metal reinforcement between so as to form a monolithic slab. It shall be laid in sections of about 9 sq. ft. area, to suit the size of the sheets of metal reinforcement. The surface shall be made true to grade and troweled to a smooth hard finish, and shall be covered completely with a coating of hot roofing pitch.

IV.—Brick Protection Course (to be Substituted for Mortar Protection Course if Specified).

Over the water-proofing layer thus formed shall be laid a protection course of hard-burned brick, except around drainage nipples, on vertical surfaces, and otherwise as called for on the drawings. Brick, unless absolutely dry and free from moisture, shall be stacked up and heated before laying, so that they shall be thoroughly dry when laid. They shall be laid on the flat directly on the water-proofing, with as close joints as consistent with provision for proper subsequent filling with pitch. The water-proofing shall be followed up closely with pouring of the joints with heated roofing pitch, which shall fill the joints completely to the top of the brick. The entire top surface of the protection course shall be mopped with one coat of hot roofing pitch at least ⅛ in. thick.

V.—Thickness of Water-Proofing on Deck-Plate Bridges.

The maximum thickness of the mortar foundation, water-proofing, and protection course shall not appreciably exceed 4 in. under any track, measuring from top of deck-plate when a mortar protection

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course is used, not more than 4½ in. when a brick protection course is used.

VI.—Time for Materials to Dry and Harden.

Mortar shall not be applied to any painted surface before the paint is thoroughly dry, or within 24 hours of the application of the last coat of paint. Pitch shall not be applied to any mortar surface unless it is absolutely dry and at least 24 hours old. Mortar shall not be applied to any pitch surface unless it is perfectly cool. Ballast shall not be laid on any mortar protection until the latter is at least 24 hours old. Tracks shall not be supported on any mortar protection until the latter is at least 7 days old.

VII.—Workmen.

Only competent workmen, especially skilled in this kind of work, shall be employed to lay water-proofing.

VIII.—Other Water-Proofing Materials.

If directed by the Engineer, asphalt or some other water-proofing compound, and other felts and reinforcing fabric, may be substituted for the roofing pitch and felt hereinabove mentioned. In such case the Company shall pay to the Contractor the excess cost, if any, of such materials, over the cost of the materials specified.

IX.—Roofing Pitch.

Roofing pitch shall be the best American, straight-run, coal-tar pitch, with a melting point of not less than 130 nor more than 140° Fahr., and shall have a specific gravity of not less than 1.23 at 60° Fahr., and an evaporation not greater than 8½% after 7 hours heating at 275° Fahr., and shall be otherwise equal to that furnished by ———.

X.—Asphalt.

Asphalt shall be pure, refined, high-grade asphalt, free from coal-tar, turpentine, or any of their products, and shall contain not less than 97% of bitumen, soluble in carbon-bisulphide; and, of the bitumen thus soluble in carbon-bisulphide, not less than 60% shall be soluble in 88° naphtha, air temperature. It shall not volatilize more than 5% under a temperature of 350° Fahr., in 7 hours. It must not be affected by a 25% solution of sulphuric acid acting upon it at ordinary room temperature for 7 hours. It shall not flow under a temperature of 212° Fahr. and shall not become brittle at 15° below zero Fahr., when spread in a layer ⅛ in. thick, on thin glass.

XI.—Other Water-Proofing Compound.

Water-proofing compounds, felts, and fabrics, other than those specified herein, shall be of quality approved by the Engineer.

XII.—Cement, Sand, and Mortar.

Cement, sand, and mortar, shall be as required by the Standard Specifications for Concrete Masonry.

XIII.—Brick.

Brick shall be dense, hard-burned, of uniform size and quality, with square corners and plane sides, and shall not increase in weight more than 10% when immersed in water for 7 hours. The brick shall not be more than 2 in. thick. The lateral dimensions shall be satisfactory to the Engineer.

XIV.—Tarred Felt.

Tarred felt shall be saturated felt weighing not less than 14 lb. per 100 sq. ft., and shall be equal to that known as ———, manufactured by ———.

XV.—Reinforced Felt.

Reinforced felt shall be equal to that known as ——— and manufactured by ———.

XVI.—Metal Reinforcement.

Metal reinforcement for the protection course shall be No. 27 gauge expanded-metal lath; No. 18 gauge 1½-in. mesh wire cloth or other equivalent reinforcement.

C. T. DeLAMERE,* Assoc. M. Am. Soc. C. E. (by letter).—The Canadian Pacific Railway Company is building a branch through the Cities of Montreal and Maisonneuve, in the Province of Quebec. On this branch thirteen solid-floor steel bridges over streets have been completed with water-proofing, and eight are still under construction. Mr. DeLamere.

These thirteen structures were water-proofed under varying weather conditions, and were exposed to severe cold (28° below zero) and extreme heat (125° air above the mastic) before the ballast was placed on the deck; thus affording an excellent opportunity to observe the water-proofing under different conditions.

Certain changes were suggested as a result of this work, and a proposed standard and drainage diagram for the unfinished work was made up, Plate VII and Fig. 20, although not yet approved. Attention is called to the following features.

1.—The introduction of a thin slab of reinforced concrete between the steel floor-plate and the membrane will act as an insulator, protecting the membrane from the violent temperature changes to which it would be subjected if in direct contact with the steel floor-plate.

2.—The membrane is laid with a steep pitch, to shed the water as quickly as possible; 1 in. in 8 ft. is recommended.

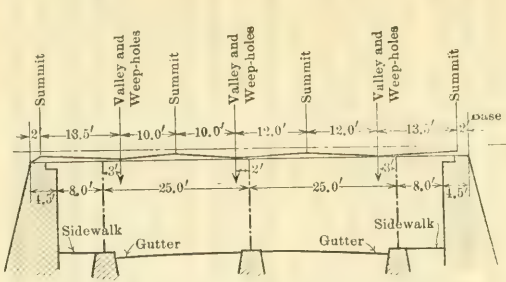
3.—The membrane should be pitched, so as to throw the water away from the angle between the floor and the side. Particular attention is paid to this feature on half-through girders with gusset-plates.

4.—Water should be removed from the bridge floor as quickly as possible, and there should be weep-holes at frequent intervals. Attempts to drain water for any considerable distance on the bridge floor or over the back of the abutments are discouraged.

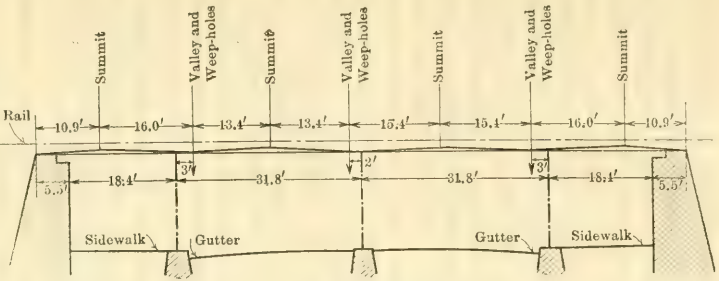
5.—Mastic protection should not be used above the ballast where it will be exposed to extremes of heat and cold and subjected to rapid temperature changes. A reinforced concrete protection is recommended for protection above the ballast. In the stringer bridges it will be noted that there is expanded metal in the concrete filler beneath the membrane, and this projects from beneath the flange angle of the girder, while the membrane is being built. After the membrane is completed, the expanded metal is bent down, and the protecting concrete is cast around it. The protection is thus anchored securely against slipping down and forming an opening, between the protection and the flange angle, which would admit snow and water. In the case of half-through girder spans, the protection is attached directly to the gusset-plates by reinforcing bars hooked through holes punched on a line with the base of rail. On these girders the joint with the steel is made tight with bitumen putty.

6.—Particular attention is called to the design of the weep-hole and grating, which is believed to have several advantages over those shown

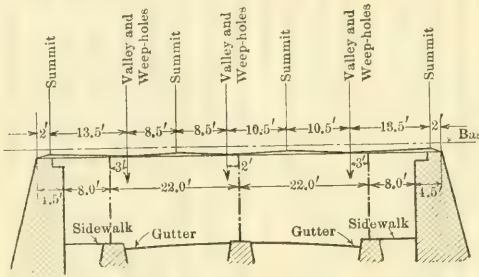
* Montreal, Que., Canada.



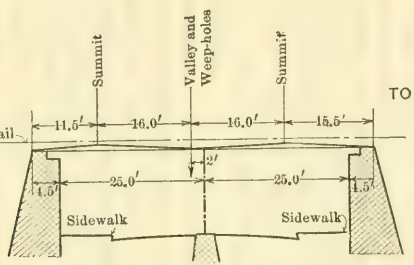
DRAINAGE DIAGRAM, BOURBONNIERE AND ORLEANS STS.
66-FOOT STREETS.



DRAINAGE DIAGRAM, PIE IX ST.
100-FOOT STREET.



DRAINAGE DIAGRAM, DES JARDINE, LA SALLE AND
LETOURNEAUX STREETS
60-FOOT STREETS.



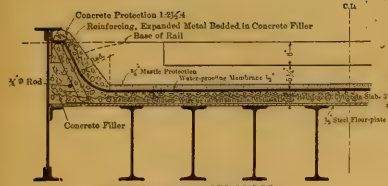
DRAINAGE DIAGRAM, CHARLEMAGNE ST.
50-FOOT STREET.

PROPOSED DRAINAGE
FOR WATER-PROOF BRIDGES
IN THE CITY OF MAISONNEUVE.
TO BE BUILT ACCORDING TO PLAN OF
PROPOSED STANDARD
WATER-PROOFING.
FORSYTH STREET BRANCH.
CANADIAN PACIFIC RY.

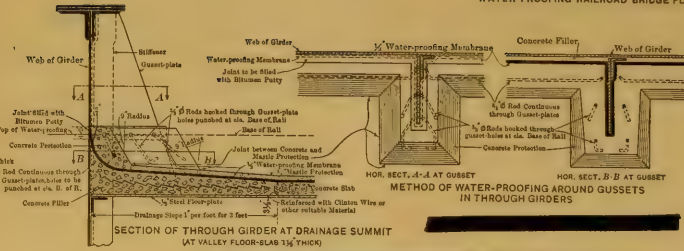
FIG. 20.

Mr. DeLamere.

PLATE VII.
PAPERS, AM. SOC. C. E.
MARCH, 1915.
DELAMERE ON
WATER-PROOFING RAILROAD BRIDGE FLOORS.

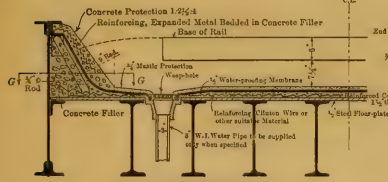


SECTION OF STRINGER BRIDGE
AT DRAINAGE SUMMIT

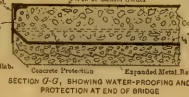


SECTION OF THROUGH GIRDER AT DRAINAGE SUMMIT
(AT VALLEY FLOOR-SLAB 3½" THICK)

METHOD OF WATER-PROOFING AROUND GUSSETS IN THROUGH GIRDERS



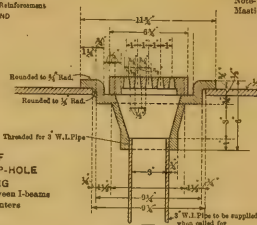
SECTION OF STRINGER BRIDGE
AT DRAINAGE VALLEY SHOWING WEEP-HOLE



SECTION G-G, SHOWING WATER-PROOFING AND PROTECTION AT END OF BRIDGE

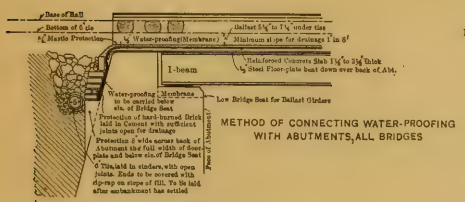
Mastic Protection to be used only on flat slopes and where covered with ballast
1 layer building paper laid dry
2 layers 16 lb. Tarred wool felt in hot asphalt cement
2 layers 8 oz. Burlap laid in hot asphalt cement
1 layer 16 lb. Tarred wool felt cemented, joints
1 layer building paper laid dry
Reinforced Concrete Slab
Floor Plate, joints covered with tarred fabric

SECTION OF WATER-PROOFING MEMBRANE AND PROTECTION ON BRIDGE FLOOR



Note-Two inches of ¾"-¾" Crushed Stone to be used next to the Mastic, Crushed Rock Ballast to be used for the remainder

DETAIL OF
CAST-IRON WEEP-HOLE
AND GRATING
Designed to be used between I-beams
1 foot 4 inches Centers


Base of Rail
Bottom of Slab
Mastic Protection
Ballast 3½" to 1'-1" under ties
Water-proofing Membrane
Minimum slope for drainage 1" in 8'
Reinforced Concrete Slab 1'-1½" to 3½" thick
Steel Floor-plate bent down over back of Abt.
I-beam
Water-proofing Membrane to be carried below sta. of Bridge Seat
Low Bridge Seat for Ballast Girders
Protection of hard-burned Brick laid in Cement with sufficient joints open for drainage
Protection 8" wide across back of Abutment the full width of floor-plate and below sta. of Bridge Seat
6" Tile, laid in cinders, with open joints. Ends to be covered with rip-rap on slope of fill. To be laid after embankment has settled
From A Abutment?

METHOD OF CONNECTING WATER-PROOFING
WITH ABUTMENTS, ALL BRIDGES

PROPOSED
STANDARD WATER-PROOFING AND
WEEP-HOLES
FOR STEEL-FLOOR BRIDGES
CONSTRUCTION DEPARTMENT
CANADIAN PACIFIC RY.

Mr. DeLamere.

in Mr. Wagner's drawings. This is flanged so that no special riveted device is necessary to support it. Circular holes are merely punched in the floor-plates, when called for. The weep-holes and gratings, if purchased in quantity, can be supplied at a low figure. They are placed by merely dropping them into the hole provided in the floor-plate. The grating is secure against overturning due to unequal pressure from the ballast.

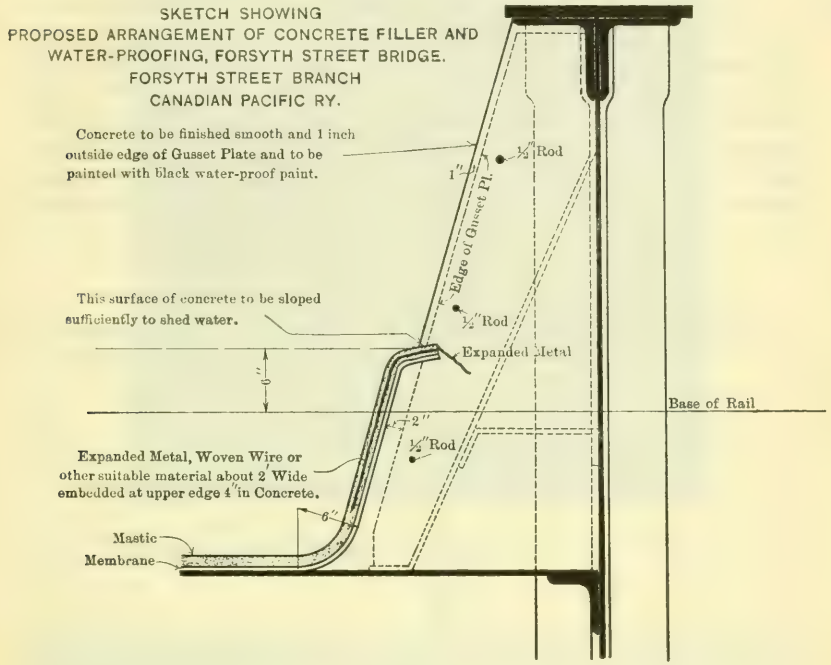

SKETCH SHOWING
PROPOSED ARRANGEMENT OF CONCRETE FILLER AND
WATER-PROOFING, FORSYTH STREET BRIDGE.
FORSYTH STREET BRANCH
CANADIAN PACIFIC RY.
Concrete to be finished smooth and 1 inch outside edge of Gusset Plate and to be painted with black water-proof paint.
½" Rod
1"
Edge of Gusset Pl.
This surface of concrete to be sloped sufficiently to shed water.
½" Rod
Expanded Metal
Base of Rail
2"
½" Rod
Expanded Metal, Woven Wire or other suitable material about 2' Wide embedded at upper edge ½" in Concrete.
6"
Mastic
Membrane

FIG. 21.

7.—The membrane at the back of the abutment is protected from shearing along the edge of the floor-plate by brick. The membrane is carried below the bridge seat level to insure against seepage, and brick protection is built up, which will allow expansion of the floor-plate, and secure the membrane from being torn by the pressure of filling and ballast.

Fig. 21 shows the method (used successfully) of securing the mastic protection to the concrete, expanded metal reinforcement being embedded in the concrete filler. Difficulty had been caused by the mastic slipping on the membrane during hot weather. In moulding this concrete filler, a slot was cast along the upper edge of the surface to be water-proofed. Expanded metal was embedded in the concrete, and

Mr. DeLamere. projected about 15 in. from this slot. The metal was bent up out of the way, and the membrane was laid running into the slot. After the first coating of mastic protection, the expanded metal was bent down and secured with a few roofing nails. Mastic was plastered over, and the expanded metal was entirely embedded in it. The mastic is thus secured against slipping down in hot weather. However, the use of mastic is not recommended for exposed work in the climate of Montreal.

Mr. Vail. JOHN JERVIS VAIL,* ASSOC. M. AM. SOC. C. E. (by letter).—In water-proofing half-through bridges in elevating the Pennsylvania Railroad at Rahway, N. J., especial pains have been taken to exclude water where ends of girders meet. At such places, where girders come together over columns, rain may enter and corrode the steel, penetrating under water-proofing, perhaps, and doing damage elsewhere, also. Fig. 22 shows the protective copper flashing devised by H. R. Leonard, M. Am. Soc. C. E., Engineer of Bridges, and R. Farnham, Jr., M. Am. Soc. C. E., Assistant Engineer of Bridges.

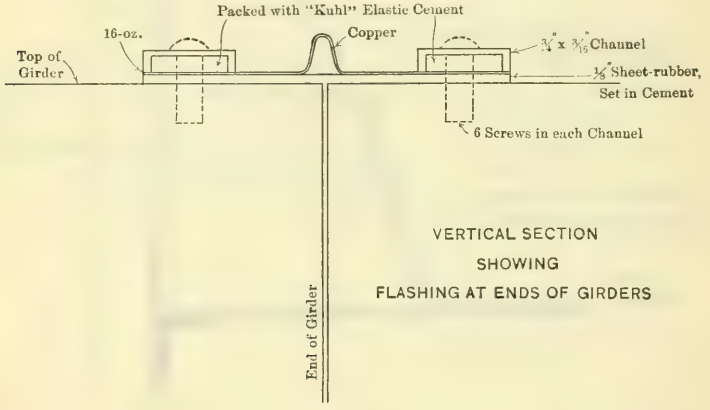


FIG. 22.

The sheet-metal, crimped over the joint at the top of the girders, is carried down the whole height of the girders, inside and outside. On the inside another jacket is placed, after the water-proofing membrane is complete, to shed water away from any break in the felt caused by contraction cracks in the masonry over the columns. To this jacket copper is soldered to form a contraction joint in the 2½-in. sheet of masonry which protects the felt, preventing the unsightly crack which may otherwise be expected there.

It is essential that the finished masonry be beveled under the top flange of the girder. The vulnerable spot in water-proofing is where the membrane is finished against the web of the girder. This attach-

* Rahway, N. J.

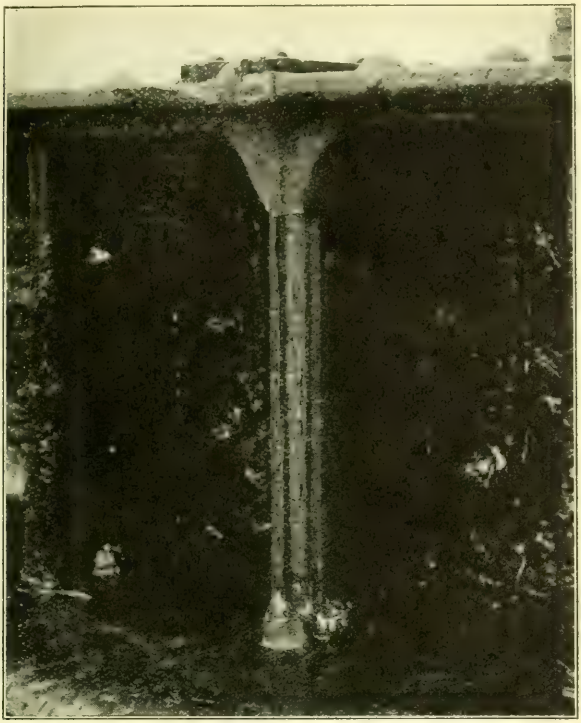
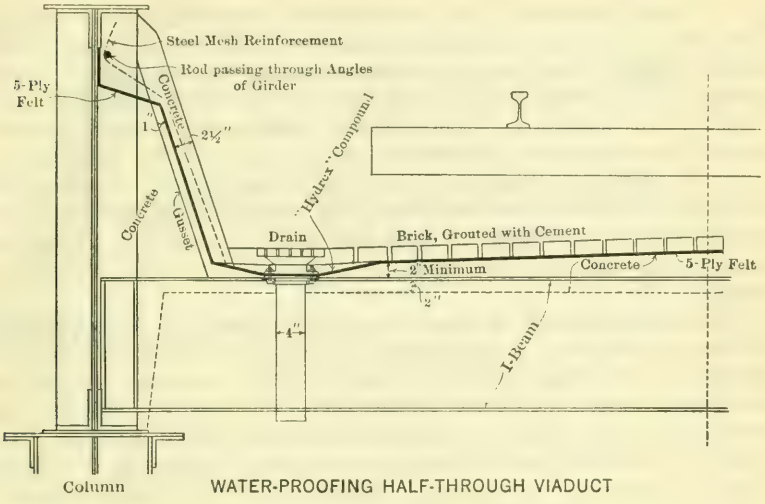


FIG. 23.—COPPER FLASHING AT ENDS OF GIRDER, TOP AND SIDE. INSIDE OF GIRDER.

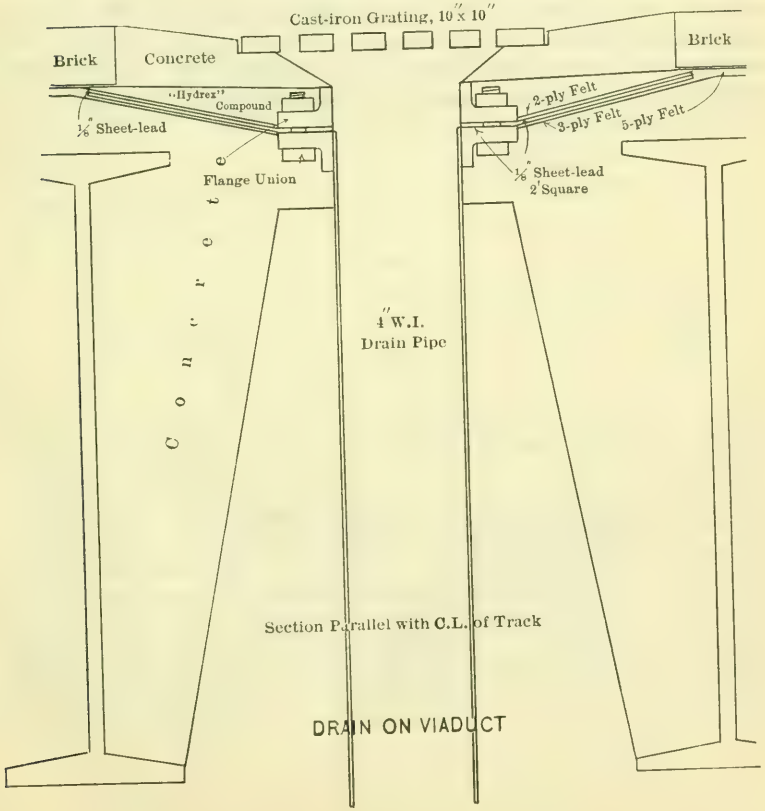
FIG. 24.—REINFORCED CONCRETE SLAB FLOOR, NOT IN CONTACT WITH GIRDERS.

Mr. Vail.



WATER-PROOFING HALF-THROUGH VIADUCT

FIG. 25.



DRAIN ON VIADUCT

FIG. 26.

Mr. Vail.

ment cannot be relied on to be water-tight, especially around angles and rivets. A bridge finished with masonry flush with the edge of the girder admitted rain water, driven by storm, into the minute space between the concrete and the under side of the top of the girder. It found its way behind the water-proofing, dripping out along the girder, under the bridge floor. A chamfer, cut as shown in the drawing, effectually shed the water, and made the bridge tight. To place the concrete as close as possible against the under side of the top of the girder, the forms are filled an inch or two above the girder. When the masonry has set, the surplus is cut away with a chisel and bush-hammer.

It would seem that if any water at all passes the water-proofing devices, the design is not a success. It cannot be known whether the members of a bridge—covered as they are with concrete—are safe or are rusted away. A bridge of the type shown by Fig. 24 has the advantage of exposing the whole superstructure for inspection. The track is carried by a reinforced concrete slab, resting on the longitudinal members of the bridge. A bridge of this kind, the floor poured in one operation, hardly need be water-proofed at all. The vertical distance from the base of rail to the clearance line over the street is 3 ft. $10\frac{3}{4}$ in., in this respect being at a disadvantage compared with a transverse I-beam floor bridge, which requires 3 ft. 0 in., or with a transverse trough-floor bridge, which requires only 2 ft. 8 in.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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NOMOGRAPHIC SOLUTIONS FOR FORMULAS OF VARIOUS TYPES

Discussion.*

By Messrs. J. O. Eckersley and Paul C. Nugent.

Mr. Eckersley.

J. O. Eckersley,† M. Am. Soc. C. E.—The speaker is of the opinion that this paper has not made it entirely clear that these so-called nomographs are nothing more or less than graphic tables which have been prepared by giving the variables in an equation successive integral values, for a considerable range, and plotting the function (with proper units) on a system of co-planar lines, approximating the intervening values of the variable by eye, instead of tabulating them in the usual manner. For example, let $4x + 5y = 60$. Tabulating, we have

x.	y.	Constant.	y.	x.
0	12	60	0	15
1	11.2	.....	1	13.75
2	10.4	.....	2	12.50
3	9.6	.....	3	11.25
4	8.8	.....	4	10.

The nomograph is Fig. 21, and the diagram, using Cartesian co-ordinates, is Fig. 22.

Fig. 21 (parallel co-ordinates) and Fig. 22 (rectangular co-ordinates) illustrate the "principle of duality", that is, a relation of lines and points such that any relation in the one gives a relation in the other by interchanging the words "line" and "point". For example, in Fig. 21 two lines determine a point, F; and in Fig. 22 two points (0, 12) and (15, 0) determine a line.

* Discussion of the paper by R. C. Strachan, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† New York City.

Mr. Eckersley.

In 1884, or about 30 years ago, this particular system of tabulation was introduced, and since then scores of writers have contributed to it. In America some use has been made of these diagrams by mechanical and electrical engineers, but they were known by other names, such as "Graphical Arithmetic", "Graphic Charts", etc. The word "nomograph", by itself considered, is a misnomer. Etymologically, it means "a written law." As a matter of fact, nomographs do not give the laws governing the function any more than a table of logarithms gives the law governing logarithms. Law, in a mathematical sense, is defined as: The rule or formula by which certain functions vary, or according to which certain changes take place, for example:

$$x = e^y$$

or $y = \log. x =$ the logarithmic curve.

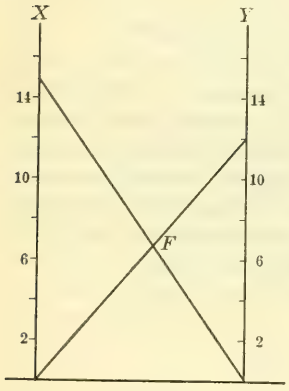

FIG. 21.

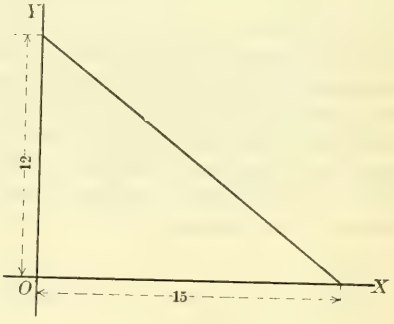

FIG. 22.

Then $$dy = \frac{1}{x}\,dx \text{ is the law.}$$

$$\frac{dy}{dx} = \frac{1}{x} = \text{tan. } \phi.$$

when $x = 1$. tan. $\phi = 1$; that is, $\phi = 45°$
when $x > 1$. tan. $\phi < 1$; that is, $\phi < 45°$
when $x < 1$. tan. $\phi > 1$; that is, $\phi > 45°$

which proves that the curve (Fig. 23) lies entirely below the tangent at $x = 1$. From $dy = \frac{1}{x}\,dx$ it is seen that when the number, x, is greater than 1, the logarithm changes more slowly than the number, and conversely.

Mr. Eckersley.

Similarly, for the equation,

$$4x + 5y = 60$$

$$4\,dx + 5\,dy = 0$$

$$\text{or, by the law,} \quad \frac{dx}{dy} = -\frac{5}{4} = -1.25$$

$$\frac{dy}{dx} = -\frac{4}{5} = -0.8$$

that is, the law gives the relation at any arbitrary point, while in general, the best that can be done with a nomograph is to determine a difference for any arbitrary values on the scales, but from the data obtained there is no assurance of what the difference would be on any other part of the scales.

As these tables depend on measurements, more than on anything else, it would seem that "metrical tables" is a far more suggestive title; and that the well-known mathematical term, "connector" would be much more appropriate than the word "isopleth." The terms "metrical tables" and "connector" suggest relevant and important ideas, as names should always do.

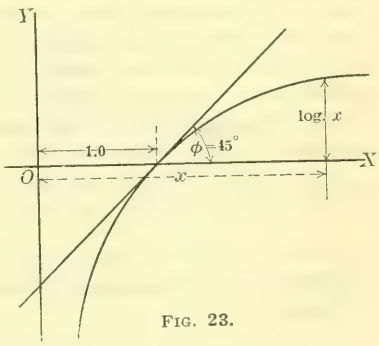

FIG. 23.

Passing to their practical value, it is noted that the method is analytical, but the conceptions and representations are purely metrical, and not to be confused with or related to grapho-statics, the difference being that the latter deduces from graphs results of the same form, by means of lines derived systematically from certain fundamental notions, for example, the funicular polygon, the reciprocal figures of Cremona, Maxwell, etc. Enough has been shown by the author to establish the claim that these tables are worthy of study, but, unless they have been studied and one has acquired the ability to select the proper scales and transform the equations into such forms as permit of this tabular plotting, it will be very much like one who reads poetry and has to spell each word; in other words, "one will not go far"; and it is inability to do those things which explains why these diagrams are not more popular.

When an engineer uses a particular formula (involving several variables) very frequently, it might be of advantage to him to prepare a table, but so many formulas depend on two variables that the Cartesian presentation is entirely satisfactory. Indeed, many are so simple that they need no diagram, *e. g.*, in the formula for pin moments, for which Fig. 5 is the nomograph, an engineer with a

Mr. Eckersley.

utilitarian bias would see that using $M = 0.1\ f\ D^3$ instead of $M = 0.098\ f\ D^3$ would induce an error of $\frac{2}{10}$ of 1%, on the safe side, which is negligible, and the calculation becomes a mental exercise, in many cases. However, it was the diversity in the methods of presentation of functions of several variables that led d'Ocagne and others to develop this uniform method, known as nomography, and any one who wishes to obtain a comprehensive grasp of the subject can refer to d'Ocagne's works, or to the great German and French mathematical encyclopedias, where he will find a systematic exposition and ample references to original memoirs.

The author is to be congratulated for this able presentation. The proofs are concise, and no doubt his paper has brought to the attention of many a very useful and convenient way of tabulating a function of several variables.

Mr. Nugent.

PAUL C. NUGENT,* M. AM. SOC. C. E. (by letter).—Mr. Strachan has covered admirably the theory of the nomographic diagram. His paper is of especial value in calling attention to some of the less familiar forms. Particularly to be noted are the diagrams in which the natural scales are used. As a somewhat different method of presentation is often of interest, the following treatment of the theory and construction of a logarithmic nomograph, similar to the one representing the Hazen-Williams formula, is here submitted.

The typical form of an equation of this kind is

$$K_q\ Q^a = K_d\ D^b \times K_h\ H^c \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which Q, D, and H are variables, and K_q, K_d, K_h, a, b, and c, are constants. A representative formula is that of Flamant, which may be written

$$Q = 0.001361\ D^{\frac{19}{7}}\ H^{\frac{4}{7}} \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In Equation (2) Q is in cubic feet per second; D is the diameter of the pipe, in inches; and H is the drop, in feet, of the hydraulic gradient per 1 000 ft. of pipe line.

Referring to Fig. 24, on the vertical line, H, are supposed to be laid off, from the point H downward, the logarithmic values of $K_h\ H^c$. It is next necessary to assume the distances, x and y. These, for convenience, may be equal. Having assumed them, draw two vertical lines, D and Q, as shown. Assuming any convenient point, Q, on the Q line, draw a straight line, RS, through this point and H. Q is the beginning or zero point of the Q line, and D, the intersection of RS with the intermediate vertical previously drawn, is the corresponding point on the D line.

To discuss first the case where the Q, D, and H factors, including their constant exponents and coefficients, are each not less than unity,

* Syracuse, N. Y.

suppose any straight-edge, $L_1\ M_1$, laid across the diagram as shown in Fig. 24. It is evident that the Q and D quantities must increase upward, and the H quantities downward, from what may be termed the "zero line", RS. Consequently, in the assumed case (all factors not less than unity and therefore all logarithms plus) $L_1\ M_1$ will intersect RS to the right of the D line, and will cut the three verticals in the points, Q_1, D_1, and H_1. Mr. Nugent.

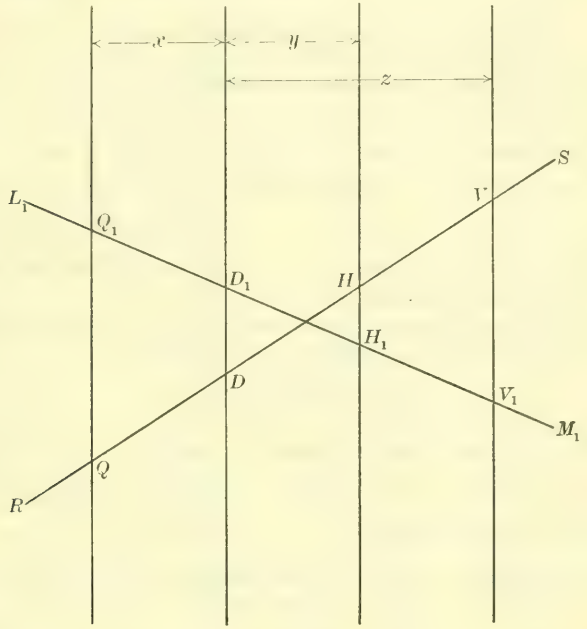

FIG. 24.

From the figure,

$$\frac{HH_1 + QQ_1}{x+y} = \frac{HH_1 + DD_1}{y}, \text{ from which}$$

$$QQ_1 = \frac{x}{y}\,HH_1 + \frac{(x+y)}{y}\,DD_1 \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Now, suppose that the scale to which the Q quantities are laid off is n_h and n_d times the scales for the H and D quantities. That is, the actual space, say in inches, on the Q scale which corresponds to the logarithm of any number as 5, will be n_h times the actual corresponding space on the H scale. From an inspection of Equations (1) and (3) it is evident that, if the nomograph is to represent the typical Equation (1), n_h and n_d, respectively, must equal $\frac{x}{y}$ and $\frac{x+y}{y}$.

Mr. Nugent.

Any convenient scale having been selected for the H line, the Q and D scales may now be determined.

The distances along any vertical, as the D line, from a point marked with a certain value of D to a value ten times as great, will be constant. These distances may be called the "secondary unit lengths" and denoted by L'_q, L'_d, L'_h. The "unit lengths", that is, the lengths corresponding to the logarithm of 10 (unity) are symbolized by L_q, L_d, and L_h.

The additional details of the construction may perhaps be best illustrated by the application of the theory to Equation (2) and its nomograph, Fig. 25. As shown in this figure, x and y are each assumed as 1.25 in. Consequently, $n_h = 1$ and $n_d = 2$.

In the formula, D occurs as $0.001361\ D^{\frac{1.9}{7}}$ and H as $H^{\frac{4}{7}}$. Q is found simply as Q. The scale assumed for the H line is 3 in. = unity. Therefore, $L'_h = \frac{4}{7} \times 3 = 1.71$ in. Similarly, $L'_q = 3.00$ in. and $L'_d = 4.07$ in. The spaces, L'_q and L'_h, are laid off up and down the Q and H lines from the zero line, RS.

Turning to the D line, we note that where it is intersected by RS, the value written for D, (D_0) must be such that

$$\log.\ \left(0.001361\ D_0^{\frac{1.9}{7}}\right) = 0$$

from which, $D_0 = 11.38$.

As shown above, the D scale is one-half the H scale, and consequently $L_d = \frac{1}{2} \times 3 = 1.50$ in. To get back to the point marked 10 on the D line, we have

$$\left[\log.\ \left(0.001361 \times 11.38^{\frac{1.9}{7}}\right) - \log.\ \left(0.001361 \times 10^{\frac{1.9}{7}}\right)\right] \times 1.50 \text{ in.} = 0.23 \text{ in.}$$

This locates the 10 point on the D line from which the spaces, $L'_d = 4.07$ in., are laid off up and down the line.

If a V line is desired, we may transform Equation (2) by writing AV for Q, obtaining finally

$$V = 0.2495\ D^{\frac{5}{7}}\ H^{\frac{4}{7}}$$

and, proceeding as above, construct a diagram based on the D, H, and V lines. More simply, however, we may write

$$Q = A\ V = \frac{\pi\ D^2}{4 \times 144}\ V = 0.005454\ D^2\ V \quad \ldots\ldots\ldots\ldots (4)$$

Assuming the V line z inches to the right of the D line, just as above, and considering the Q, D, and V lines, $n_v = \frac{x}{z}$ and $n_d = \frac{(x+z)}{z}$. Q is involved in Equation (4) just as in Equation (2), and we may therefore assume the plotted Q line as a basis for the V construction.

Mr. Nugent.

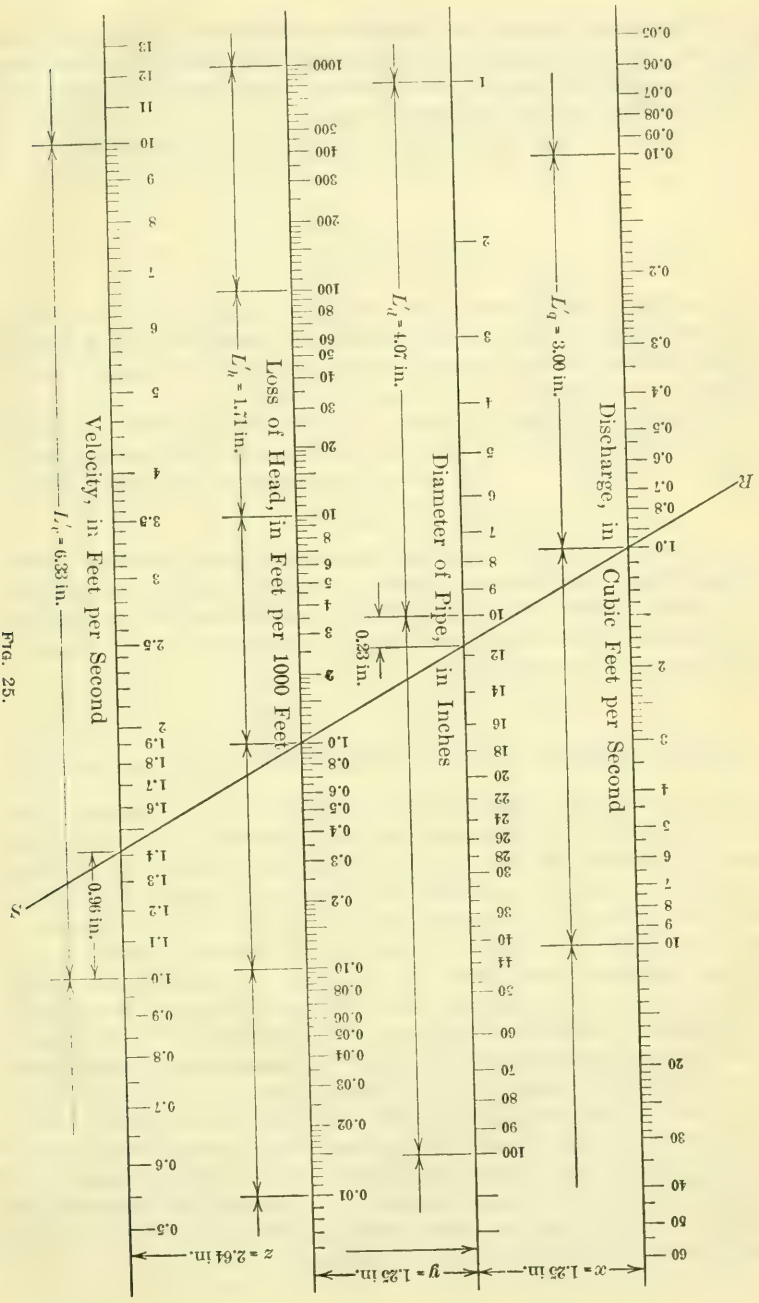

FIG. 25.

Mr. Nugent.

The coefficients, K_d, etc., in the general Equation (1) do not enter into the determination of the secondary unit lengths. These are matters of the exponents, and, having determined L_d, $L'_d = b\,L_d$. Thus, considering the Q, D and V lines,

$$L'_d = b\,L_d = b\,\frac{L_q}{\frac{(x+z)}{z}}, \text{ from which}$$

$$z = \frac{L'_d\,x}{b\,L_q - L'_d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Using the values above given, from Equation (5), $z = 2.64$ in. Note that what has been done involves the assumption of a new L_d for the D line. Since the unit lengths have not been shown on the diagram, no confusion will be caused thereby.

The determination of the value, D_0, to be written at the zero point of the D line, has involved both the coefficient and exponent of the D factor in Equation (2). The general condition to be satisfied here is

$$\log.\ K_d + b(log.\ D_0) = 0 \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

From Equation (4), $b = 2$, and we have already found that $D_0 = 11.38$. Making these substitutions in Equation (6), we obtain $K_d = 0.007729$.

It now becomes necessary to write Equation (4) in such a way that this special value of K_d appears as the coefficient of D^2. Equation (4), therefore, is transformed to

$$Q = 0.007729\ D^2 \times 0.7057\ V \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

We next have $L_v\,(= L'_v) = \frac{L_q}{n_v}$, from which, with the foregoing values, $L'_v = 6.33$ in. The value to be written at the intersection of the RS and V lines and the distance from this intersection to the end of the nearest L'_v space are determined as with the D line. V_0 is found to equal 1.417, and the distance up to the point marked l is 0.96 in. From this point the L'_v spaces are laid off upward and downward. Less than two such spaces are needed in the present diagram.

For subdividing the spaces, L'_q, L'_d, etc., the construction shown in Fig. 26 may conveniently be used. A right triangle is drawn with a base of any convenient length and an altitude equal to the longest of these spaces, in the present case L'_v. On the hypotenuse are chosen points at distances from the base equal to the various lengths to be subdivided and also to the unit length of an ordinary slide-rule. Lines are drawn from these points perpendicular to the base, and the line corresponding to the slide-rule is divided as on that instrument. A series of lines through these points of division and the vertex of the triangle will subdivide the other lines proportionately. The subdivisions are finally transferred to the nomograph.

Mr. Nugent.

To consider the case where one or more of the various factors are less than unity, Figs. 27 to 35 show the nine possible positions of the straight-edge, or isopleth, $L\ M$. With each scale (Q, D, H, or V) increasing in the same direction both below and above the line, RS, the numerical value of the logarithm of any quantity read off on

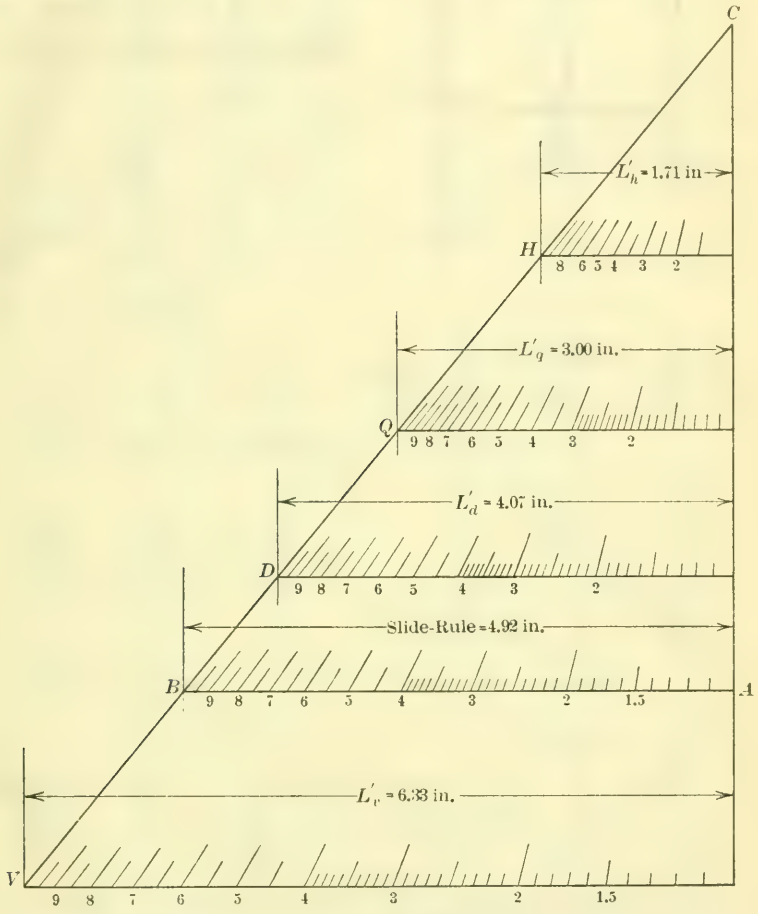

FIG. 26.

the diagram is always proportional to the distance between the point read and the RS line. In the general Equation (1), in solving for the Q factor, if its logarithm is plus and the logarithms of the other two factors are one plus and the other minus, the logarithm of the Q factor will be numerically equal to the difference between the other

Mr. Nugent.

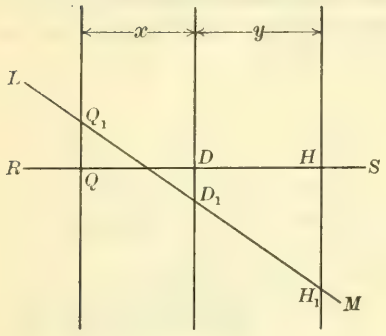

FIG. 27.

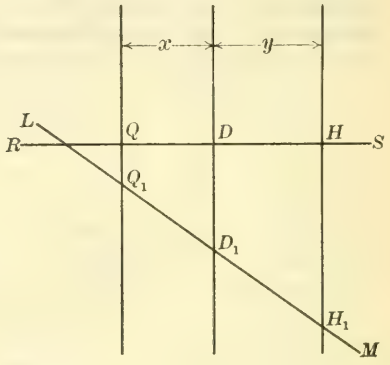

FIG. 28.

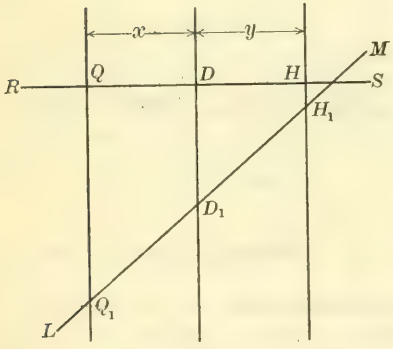

FIG. 29.

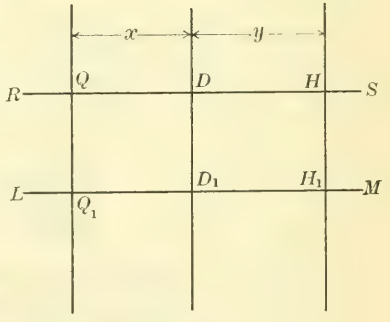

FIG. 30.

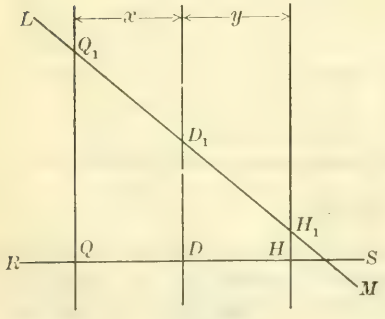

FIG. 31.

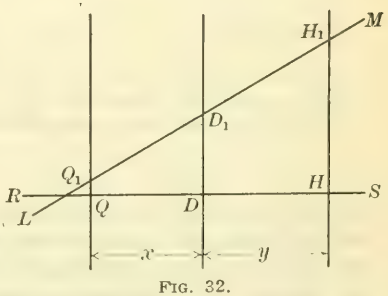

FIG. 32.

two. This corresponds to Figs. 27, 31, 32, and 35. The same thing is true if the logarithm of the Q factor is minus and the other two logarithms are one plus and the other minus. This corresponds to Figs. 28, 29, 30, and 33. If all the logarithms are negative, the logarithm of the Q factor is numerically equal to the sum of the other two; this corresponds to Fig. 34. This last case is entirely similar to that already discussed, where all the logarithms are positive. Cases where one or more of the logarithmic factors become equal to zero, may be treated as special cases under some of those previously mentioned. Mr. Nugent.

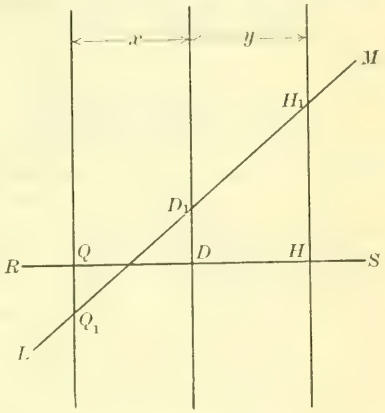

FIG. 33.

To justify the diagram for the cases shown in Figs. 27 to 33 and 35, it is necessary to show for each only that

$$QQ_1 = \pm (n_h\, HH_1 - n_d\, DD_1).$$

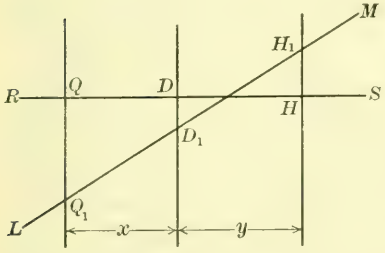

FIG. 34.

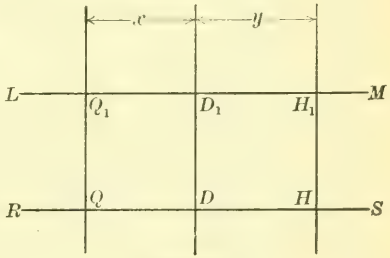

FIG. 35.

With Figs. 27 and 33 we may write

$$\frac{QQ_1 + HH_1}{x - y} = \frac{QQ_1 + DD_1}{x},$$

from which, $\quad QQ_1 = \frac{x}{y}\, HH_1 - \frac{(x+y)}{y}\, DD_1 = n_h\, HH_1 - n_d\, DD_1,$

and with Figs. 28 to 32 and 35,

$$\frac{QQ_1 + HH_1}{2}\,(x - y) = \frac{(QQ_1 + DD_1)}{2}\, x + \frac{(DD_1 + HH_1)}{2}\, y.$$

from which, $QQ_1 = \frac{(x+y)}{y}\, DD_1 - \frac{x}{y}\, HH_1 = n_d\, DD_1 - n_h\, HH_1.$

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PAPERS AND DISCUSSIONS

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RIVERS AND RAILROADS IN THE UNITED STATES

Discussion.*

By Messrs. F. Lavis, H. Burgess, and H. T. Pease.

F. Lavis,† M. Am. Soc. C. E.—This paper calls very timely attention to one of the problems of the economics of transportation which is often misunderstood or entirely ignored. This misunderstanding or ignorance has resulted in the expenditure of a great deal of money without adequate benefit, and, not only has this been the case in the past, but it still continues. Mr. Lavis.

Lately, however, there has seemed to be a growing sense of the wastefulness of the so-called "pork barrel" methods of apportioning the expenditures for the improvements of rivers and harbors, though there is still far from a general appreciation of the necessity of the study of this whole question from the standpoint of true economy.

Our National policies in these matters seem to be too often developed in a generally haphazard manner, instead of being founded on sound economic laws; and, in the general development of the whole resources of their countries, certain European nations seem to be guided by more orderly processes of thought and action than we are; but, even in Germany, the greatest exponent of order and system, it is by no means a fact that the development of its waterways has resulted in unqualified success in the attainment of low costs of transportation thereon. It has frequently been assumed that this is the case, but it has been shown‡ that in Germany, as elsewhere, many of the items which should be included in the cost of water transportation

* This discussion (of the paper by William W. Harts, M. Am. Soc. C. E., published in January, 1915, *Proceedings*, and presented at the meeting of February 3d, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

‡ "Waterways and Railways," by H. G. Moulton.

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have been omitted. Such success as has been attained, however, has been largely due to the fact that the matter of deciding on the projects on which public monies shall be spent has been generally left to a comparatively small body of men of training and experience in the particular class of work or problems involved, whereas, in the United States, this is generally decided by popular vote.

It is probably true that, in the development of their inland waterways, some of the European nations have attained a greater measure of success than we have, but in economic progress in the general realm of transportation, including that by railways, it is undeniable that we have far out-distanced the rest of the world. This success, generally speaking, has also been the result of policies formulated by a few men or by small groups who have brought our railroads to the high plane of efficiency on which they rest to-day. The word "rest" is used advisedly, as apparently the initiative which has achieved these results is now to be held in abeyance by the passing of the control of our transportation systems into the hands of those charged by the State with their regulation, and who have not the personal incentive to strive "to make a dollar earn the most interest."

Under our form of government it is inevitable that the final decision in regard to the expenditure of money for public improvements must rest on the approval of the people as a whole, and it is perhaps too much to expect that any large proportion of them can be so educated as to be able to form independent judgments on all matters of this kind, and especially is this so in the realm of economics, but, as a whole, they are generally quick to grasp the essentials if these are clearly presented to them.

The State of New York is just about completing a new canalized waterway from the Great Lakes to the navigable waters of the Hudson River at a cost of approximately $130 000 000, or more. The construction of this canal was authorized by vote of the people of the State, only a few years ago, and apparently they were influenced by two arguments, both of which, in the speaker's opinion are fallacious. These arguments were:

First, that by providing competition the canal would compel the railroad lines to keep freight rates down.

Second, that the actual cost of transportation would be less by the canal than by railroad.

It does not seem necessary at this time to go into much detail to show how these arguments fail. The theory of competition is scarcely tenable, in view of the almost absolute control of railway rates by both State and Federal regulation. The actual cost of transportation by canal is only less when the interest on the investment in the canal and the cost of its administration and maintenance are ignored.

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Taking into consideration the interest on the investment and the cost of maintenance, it has been shown* that the cost of transportation on the Erie Canal is about 8.6 mills per ton-mile, and this for low-grade, bulk freight alone, whereas, the receipts of the New York Central Railroad for all classes of freight were only 6.2 mills per ton-mile, indicating a considerably lower figure even than this for the low-grade, bulk freight.

It is also shown that the cost of construction of the old Erie Canal was not less than about $165 000 per mile, and the new canal will cost double that, which may be compared with the capitalized value of the New York Central Railroad at about $180 000 per mile.

Professor Engels, an eminent German authority, has stated† "that to improve a German river involved an expense equal to that of building a double-track railroad on easy gradients through a mountainous country", that is to say, from $100 000 to $150 000 per mile.

Even in regard to the Great Lakes of Canada, where water transportation, except for the obstacles of the various locks, is somewhat on a par with ocean transportation, *Sir* Sandford Fleming, M. Am. Soc. C. E., a well-known authority on transportation, speaking of the then proposed Grand Trunk Pacific Railway, said:‡

"I have faith in an all-rail means of conveying the products of the farm to the seaboard * * *. Do not lose a day in proceeding with the location of a railway that will carry grain from the distant prairie fields to Quebec cheaper than by any other route whatever. * * * [The railway route] would be entirely free from the abnormal haste incident [to navigation on the Lakes] during the short period after harvest when it is possible to despatch the year's crop to market."

The question of the value of waterways for the purpose of providing competition for the railways is very ably discussed§ by M. Colson, an eminent French authority, who uses as an illustration a specific case in France where the relations were apparently somewhat similar to those of the New York Central and the Erie Canal. The argument is paraphrased to fit this latter example, with which we are all familiar. It is argued that, if lower rates were the desired object, there is probably no doubt that if the New York Central had been presented with say one-half the cost of the Barge Canal, and half the estimated annual cost of administration and maintenance, it would have been glad to guarantee a freight rate of 2 mills per ton-mile (the actual net transportation cost on the canal) on such items of bulk freight as may be expected to pass through the canal and during the season the

* "Cost of Transportation on the Erie Canal," *Bulletin*, Bureau of Railway Economics, 1911.

† *Engineering News*, January 21st, 1915, p. 116.

‡ *Engineering News*, November 5th, 1903, p. 416.

§ *Bulletin*, International Railway Congress, November, 1913, p. 958.

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canal is open. The lower rates would thus be secured and the State would save many millions of dollars.

Probably, however, there would be a great outcry and protest against subsidizing a particular railroad to obtain a reduction of rates in a certain local section, without obtaining the same reduction in the rest of the country or State; yet this is really what is done, except that the cheap rates are obtained for a certain small part of the population by spending the money for a canal rather than by giving it to the railroad.

The importance of the study of the questions raised by the author is clearly apparent, especially in view of the existing financial and economic crisis. All who have broad knowledge of the newer civilizations, that is, those of North and South America, as well as Africa, Australia, etc., cannot but be impressed with the tremendously large number of important things to be done, and the difficulty of getting money enough to carry out even a small part of them, including often those of considerable merit. The Balkan War and the present European War, so far as their effects have any bearing on the financial situation, are but the culmination of a vast and far-reaching crisis in the affairs of the world produced by the great demands for capital for legitimate and urgently required developments of new countries, and the totally inadequate supply. Therefore, it is doubly necessary now that the greatest care be exercised in selecting for development only those projects which are sound when judged from the broadest economic viewpoint.

The author has pointed out that, in technical skill, our engineers are quite the equals of any others. The speaker believes our Army engineers and many others are equally alive to the importance of the study of the commercial value of the projects presented for their consideration, but they are not always consulted in regard to this latter aspect, and often, if they are, their advice is ignored if it does not agree with political plans. It is also only too true that many engineers are so engrossed with the technical details of their work that they lose sight of its broader economic aspect, and it is well that our attention should be called, at this time, to this often very important phase of our work. In converting the forces of Nature to the use and benefit of mankind we must not lose sight of the fact that the extent and value of this use or benefit is most often measured in terms of dollars.

The present war has served to focus the attention of many of our people on our foreign trade, and, as usual, just as happens after an accident to a steamer or a train, the great mass of the people wakes up with a shock, the Public Service Commissions rush to make orders, the Legislatures and Congress pass or attempt to pass new laws, and now a great foreign commerce is to be created out of nothing in 24

hours. The speaker is in entire agreement with the conclusions of the author that it is time that we awoke to the wastefulness of much of the expenditure for improvements of rivers, and to the desirability of applying a larger proportion of the money toward increasing the facilities and depth of water in our ocean ports. He has not the faintest expectation that our foreign trade is going to jump into existence at once, and particularly not on account of the war, but it is going to grow because of economic causes, which had their origins long before there was a thought of war, at least in our minds. Mr. Lavis.

We have heretofore been exporters principally of pastoral and agricultural products, and with these we have paid our debts abroad, that is, the interest on borrowed money, and for the luxuries and other goods we have bought for import. Our manufacturing, and therefore, necessarily food consuming, population, however, has been steadily growing, our surplus production of food available for export is getting less, and we must export manufactured articles to make up the deficiency. This economic condition has been growing visibly for the past two or three decades, and has been plainly foreseen by men of large vision, but the country and Congress have just awoke, and, as usual, the true economic laws governing the situation are lost sight of in the excitement of trying to do something right away, instead of utilizing the brains of trained and experienced men to develop policies based on right principles and to carry them through.

The development of our ports must be planned on an adequate basis to take care of a large overseas commerce, and our interior lines of communication must be developed to carry our products from one part of the country to another at the lowest possible cost per ton-mile. In the present state of the art, with possibly a very few exceptions, in respect to certain natural, deep, inland waterways, the railway is the best and cheapest means of inland transportation of which we have knowledge, and unless there is a distinct revolution along lines as yet unthought of, is likely to remain so for some time to come.

The author has referred to the three stages in the economic development of inland water routes, the first before the advent of the railroads, when they provided practically the only means of access and when, owing to the rapid growth of the country, they were prosperous; the second, the era when the railroads, in competition, almost entirely superseded water transport, and then thirdly, looking forward to a time when the waterways of the country may be utilized, to some such extent as they are in many parts of Europe, as almost equal in importance to the railways as lines of transportation for low-grade freight.

Of course, we cannot say that this third era will never develop in this country, but to the speaker it seems somewhat improbable that it will, if the development of the art of transportation by railway continues in the future to any such extent as it has in the past. In

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Europe the railways have never entirely superseded the rivers and canals, or offered the effective competition that they have in America, for it must be remembered that a great many of the mercantile and industrial centers of Europe were built up and established along the water routes long before the advent of railroads. Of course, many of the largest of the modern industrial and manufacturing enterprises have grown up coincidentally with the railroads, but the majority of them are only developments from those of an earlier period, built up alongside navigable rivers and canals.

Another point of considerable difference, is that in Europe, society and the industries, having been organized before the advent of the railroad as the primary agency of transport, were developed as a number of comparatively small self-sustaining groups, that is, self-sustaining so far as concerns the necessities of life. In the United States, however, the development of the railway as an efficient and cheap means of transport, reaching all parts of the country before the development of the principal industries, has enabled these latter to be located where the soil, climate, and other conditions were best suited to them, and then to distribute the finished product to the point of consumption.

In the United States the country has been developed by the railways, and though many industrial establishments have been located so as to enjoy the benefits of both rail and water transportation, the railway is always of paramount importance. In spite of some adverse criticism of some details and of some of the figures used, the statement of James J. Hill, F. Am. Soc. C. E., that, in comparison with the rates of wages, the cost of freight transportation on the railways of this country is only from one-third to one-quarter as much as in Germany, is undoubtedly substantially correct. Taking into consideration, therefore, the limitations of flexibility of water transportation routes, the necessity that business shall come to them, and that they can seldom, if ever, be carried to the business, the fact that some cargoes, notably coal and grain, deteriorate from wet and dampness, that there are other inherent difficulties in connection with inland water transport, increased hazard, and insurance rates, and considering the efficiency of our railroads, there seems little reason to expect now that there will be any general development of fluvial or canalized inland transportation, unless there is a revolution in methods entirely beyond any present expectations, and which, therefore, it is useless to attempt to provide for.

The development on a large scale of our foreign commerce is inevitable if we are to progress as a nation in the future in any degree commensurate with our development in the past. To handle this foreign commerce, we must have adequate ports and harbors and an adequate and efficient system of transportation routes leading to these

ports from all sections of the country. The railroads furnish the most efficient, that is, the best, cheapest, and quickest, means of transport we know of to-day, or which we can now imagine for the future; the logical developments, therefore, of the immediate present, and of the near future, are those calculated to increase the capacity and usefulness of our ports and of our railroads. Mr. Lavis.

The foregoing is all more or less general, and the speaker would like to call attention to two specific problems, which have come under his observation and may serve to emphasize this necessity of developing a broad economic viewpoint in regard to the larger problems of transportation.

He was very much interested a few years ago in a study of the probable lines along which the development of the railway systems of the northern part of the Argentine would be carried out, and how this development would be affected by the value of the River Parana as a transportation route. The Parana is one of the great rivers of the world, and easily comparable with our own Mississippi, and like it heavily silt bearing.

An inspection of the map will show the general location. The Parana is navigable for ocean steamers (20 ft. draft) as far north as the City of Santa Fé. It has been proposed from time to time that it might be made navigable as far as the confluence of the Paraguay and the Alto Parana, at which point are the Cities of Corrientes and Resistencia. This point is now reached by flat-bottomed river steamers of the ordinary type, and at certain stages by keel ships of 10 to 12 ft. draft. The question which presents itself is in regard to the development of the country in the inverted triangle which has Santa Fé at its apex at the south, the Parana River as one long side on the east, the line from Santa Fé through Tucuman and Embarcacion as the western side, and the Pilcomayo River, on the northern boundary of the Argentine, as the base. This area is generally known as "The Chaco", the distance from Santa Fé to the northern end of it being approximately 1 000 km.

The Government of the Argentine has started the development of this area at its northern end by the construction of two lines of railway running a little north of west from two points on the Parana, Formosa and Resistencia, apparently on the theory that the products of the region will be brought to the river and so by the fluvial route to Buenos Aires, the capital. The products of this country will be principally agricultural and pastoral, requiring cheap transportation to move them. The speaker came to the conclusion that, if an efficient system of railway transportation were developed from the City of Santa Fé spreading out in a fan shape over the area as the country developed, and probably extended north of the Pilcomayo into Paraguay and Bolivia, in spite of the existence of the semi-navigable river, the

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products of this region could be brought to Santa Fé, and if necessary south of it, by this means, more cheaply than by any other, and that there would not then be any incentive to attempt the development of the river route for ocean-going vessels to points above Santa Fé. At present it takes from one to two weeks to reach the center of "The Chaco" from Buenos Aires, if dependence is placed on the fluvial routes to reach these Government railways. The same length of line built due north from, say, Tintina, would bring this section within less than 48 hours of the Capital. Of course, there are many other factors affecting the situation, but the general problem is here outlined as a rather interesting example of those which engineers are sometimes called on to solve, which are rather outside of mere technical routine.

There is another large transportation project in which nearly all the countries of North and South America have been interested from time to time, the true solution of which, it seems to the speaker, is quite the opposite of those previously referred to, that is to say, that the railroad does not and, for a long time to come, would not afford as adequate or efficient transportation service as the water routes. The project is that of uniting all the countries of North, South, and Central America by the so-called Pan-American Railroad. Advocated first by the Pan-American Congress of 1892, which resulted in some extensive surveys, or rather reconnaissances, it has been fitfully kept alive, and we are told from time to time of the progress made toward its completion.

There is now communication from Canada, through the United States, to the southern border of Mexico. There connection is possible with the railway system of Guatemala, but this latter is of different gauge. At the southern end, both the Argentine and Chilean lines from their respective capitals, Buenos Aires and Santiago, except for a small gap in Bolivia near the Argentine border, have lines reaching to Bolivia and thence to Peru. The through routes, however, involve several changes by reason of breaks of gauge and it is interesting to note that these international lines where they have been built along the route of the proposed Pan-American Railroad, and without other particular purpose, are all operated at a loss.

Groups of enthusiasts still gravely predict and advocate the completion of the line between Peru and Guatemala, for no imaginable reason except the purely sentimental one of being able to say that there is a continuous line of railway uniting all the countries from Canada and the United States to Chili and the Argentine. Apparently, the fact that there are almost as many gauges as countries does not seem to bother them at all. There is not anywhere along this route nearly enough business in sight, either freight or passenger, to warrant the construction of even a comparatively cheap road, and this proposed line, throughout nearly its whole length, would involve some of the

heaviest and most difficult construction, in regions ranging from the densest tropical forests to altitudes of 12 000 to 14 000 ft. above sea level. That the railway lines of each country will eventually connect with each other, there is little reason to doubt, as each country develops its railway system and extends its lines to all its boundaries; but, as a through transportation route, it is entirely impractical. Generally speaking, both passengers and freight can be carried much more comfortably, expeditiously, and cheaply by ocean steamers, and if any money is available at any time in the proximate future in any of these countries, there are many other routes along which the construction of railways is far more urgently needed than that selected for the Pan-American lines. The continued advocacy of this scheme, mostly by people from the United States, seems therefore to be undesirable from the standpoint of true economic benefits to the countries concerned. Mr. Lavis.

We are beginning to realize the necessity for the conservation of all our resources of every kind, and none other is so much in need of conservation to-day as capital. As engineers, we can do our country and those dependent on us for advice and counsel no better service than to protest against wasteful expenditures for uneconomic projects, and use our whole influence to advise against any which, after careful consideration, seems to be economically unsound.

H. Burgess,* M. Am. Soc. C. E. (by letter).—The author has given those reasons for the decrease in tonnage on interior waterways which appear to be the most important; but an additional cause may be mentioned, and that is the more favorable freight rate charged by railroads on any portion of their lines for which there is a competing water route. The result of the lower rates for towns connected by a water route is that "inland" towns must pay a portion of the rail transportation charge on freight received by river towns. These latter, therefore, receive the superior facilities offered by the rail route over carriage by water at practically what such freight would cost if carried by boat, and these superior facilities are obtained at the expense of the "inland" towns. Mr. Burgess.

A railroad must collect a certain sum from its freight business for operating expenses and for return on investment, and there results an average charge per ton-mile; and if the ton-mile rate for portions of the line competing with the water routes is lower than this average, the rate must be above the average on other parts of the railroad system. The matter is complicated further, however, by the fact that this permitted lowering of the correct ton-mile charge actually succeeds in securing a greater total tonnage, with a lower average cost per ton-mile of freight carried; and, further, by the fact that water competition frequently means also either lower line grades or, for the

* Major, Corps of Engrs., U. S. A., Nashville, Tenn.

Mr. Burgess.

same grade, a lower cost of construction per mile of road, with corresponding lower cost per mile for haulage or for the interest charge. These facts are probably responsible for the Interstate Commerce Commission permitting the railroad to collect from the inland shipper part of the freight charges which should be paid by the merchant of the river town. It would be quite difficult to arrive at a correct adjustment of the charges per ton-mile between the sections of the road which have water competition and those which have not, but there is little doubt that at present the inland town is paying for facilities furnished to the river town, and that the railroad is carrying freight destined under proper freight rates for the river route. It is not forgotten that many railroad men maintain that railroads can actually carry freight at a lower cost between terminals (not per mile, the rail route being much more direct as a rule) than is possible for the river route; but this position does not seem to be consistent with the fixing of lower than average freight rates where there is water competition. It is not probable that the railroad would propose its rates to be any lower than would be required to take away the business from the water route.

It is not easy to suggest an equitable remedy for this condition; but as long as the railroad is permitted to urge water competition as an excuse for rates below the average, the boat lines can expect to carry only a small portion of the tonnage which, under normal conditions, would fall to them, and the inland town can expect to be unfairly dealt with as to freight rates for benefits which are received by the river towns.

The author has mentioned the great improvement in facilities offered to shippers by the railroad in the past few decades, but has hardly brought out with sufficient emphasis the relatively slight increase in the facilities offered by the boat lines. Relatively, the water route has fallen far behind, in spite of the fact that both as to depth and width of channels the interior streams have been improved considerably since the date when the rail and the water route were about on a parity as to advantages to the shipper. This falling behind is due chiefly to the failure to push to quick completion the improvements of streams, a failure due, in turn, to the scattering of annual expenditures on many streams, instead of concentrating the available money on a few until the work on them is completed, and then following with the improvement of others. A stream which is improved in disconnected sections is little better as to facilities for navigation than in an unimproved state, and is very much like a railroad which is completed in detached sections. Or, possibly a better comparison would be with a railroad, the roadbed of which during from 4 to 6 months per year was interrupted at intervals with submersion by overflowing streams, providing continuous traffic facilities for from 6 to 8 months per

year, and only for local traffic during the remaining months. A stream being canalized for barge traffic is not available for such traffic throughout the year until the entire improvement is completed, although, at times of high and mid-stream stages, the barges may be moved from end to end of the stream, while during the low-water season only local traffic is possible. Mr. Burgess.

Taking a specific case, the Ohio River might be compared with the trunk line of a railroad, and its navigable tributaries to the branch lines serving to feed freight to the main line. Here, due to the scattering of appropriations for river improvements, the trunk line is usable for only a portion of the year, while some of the branch lines, as the Kanawha, the Kentucky, and the Green, are at all times available for traffic, and other branch lines, like the Cumberland and the Tennessee, have a year-round traffic only for disconnected sections. Because of these conditions, and of the declination of railroads to make joint rates over land-water routes and to interchange freight with boat lines, it is not surprising that commerce on these streams is confined principally to the local movement of products, the origin and destination of which are on the river banks. What railroad system would be able to show high traffic development under conditions of complete interruption of traffic for long periods each year on its main line and most important branches, especially if other carriers would not agree to the interchange of freight on a reasonable basis? Under authority of the Panama Canal Act, the Interstate Commerce Commission can now make through rates for part rail and part water hauls; and interchange of traffic between rail and boat lines can be expected when the navigable condition of any stream is such as to warrant the exercise of the authority granted to the Commission.

When the improvement of an entire river system has been completed, with its trunk line and feeders reaching the important shipping points in its basin, and when the Interstate Commerce Commission has abolished the inequitable traffic rates heretofore permitted where there is water competition, and has directed through traffic arrangements for rail-water routing of freight, then there will undoubtedly be a considerable increase in the tonnage carried on the waterway, even without awaiting the "third stage" mentioned by the author; but whether, during his "second stage", the tonnage carried will be sufficient to justify the cost of the improvement of the streams, is another matter. The completion of the improvement of the river system and the regulation of freight rates are questions for the Law, and the beneficial results which will ensue is a question for the Prophets. In the absence of a true test, under conditions reasonably favorable to the interior waterways, there will remain great differences of opinion as to the utility of river improvement.

Mr. Pease.

H. T. Pease,* Assoc. M. Am. Soc. C. E. (by letter).—There is much to be said in favor of the author's argument for economy in rivers and harbors appropriations. For one thing, the political "pork barrel" feature should be eliminated. This is aside from the author's argument, but, nevertheless, has had to do with making the figures for such work as large as they are. However, omitting this feature, the argument still stands that much has been spent unwisely and prematurely for river and harbor improvement and, without doubt, much more might be accomplished with equal appropriations, if all the economic as well as purely engineering features were more judiciously analyzed before such appropriations were made.

There is no question that some waterways can never compete with railroads in the same territory. This is probably more nearly true of inland waterways than of those leading directly to the coast. However, the economic features of each case are the determining factors. The problem is to read them aright. Take, for instance, the Great Lakes: The tonnage on this great interior waterway is not decreasing, and the reasons are plain. The greatest wheat area of North America is at the head of the Lakes, and they lie in a direct line between that territory and the chief markets for wheat, the Eastern States and Europe. The wheat goes this way in spite of the fact that it has to be transhipped several times *en route*. Again, the greatest iron deposits of the world lie at one end of this waterway and great deposits of coal at the other. Therefore, the iron takes this route to the coal. These, however, are not the only determining features; the Lakes are broad and deep, and the draft and tonnage of Lake boats are almost as unlimited as those of sea-going vessels; moreover, the value of this route has been demonstrated, so that terminal facilities equal to those of any railroad have been provided.

The case of the Ohio River shows a contrast, both in economic features and in results. The Ohio runs the wrong way. If this river headed at Cairo and emptied into Delaware Bay, it would be to-day one of the principal routes of transportation of America. As it is, it heads in a manufacturing district and flows toward an agricultural district, and a comparatively small market for its manufactures at that. Agricultural products are heavier and more bulky than manufactures, and, in a scheme realizing the greatest economy, should move down stream, while the smaller cargoes of manufactured goods should move up stream.

The requisite for greatest economy in river transportation is that the agricultural and mining districts should be along the upper reaches of the river, and the manufacturing districts should lie along the lower portion, the port for foreign commerce being at the mouth. Gravity is an important factor where it can be utilized, for it is cer-

* Deer Park, Wash.

tainly cheap. Analogy to the modern factory may be drawn: The raw material goes in at the top, and gravity is utilized as far as possible. Mr. Pease.

The one big article of commerce which the Ohio could transport to advantage is coal, but the coal market is also in the other direction, to the north and east. It should be admitted, therefore, that at least a part of the great expenditure on the Ohio has not been warranted or is premature, and the same conclusion follows in the case of many other streams, notably the Mississippi.

An examination of the latest factors which will tend to change conditions may show a future for some of the streams at least, and give some assurance that all the work done on them has not been lost.

The Ohio and Mississippi are in the same category at present. There is no market farther down to be reached for the products at the head, and the route is not direct enough for these products to their present market, which is principally the Northeastern States and, in a smaller way, Europe. The great future market, however, will be the lower Mississippi Valley itself. It is the greatest potential area for the production of raw products in North America, and the future will see the river lined with factories from St. Louis to the mouth. Then the Ohio will ship its coal down stream—there is little water power down there—the Northwest will send down its grain to feed the ever-increasing population, and the manufactured products will in part go back up the river and in part tranship for South America and Europe. Nor will it be simply a case of over-congestion of population forcing a part of the traffic to the river, but rather that the river itself will outdo all competitors in transporting such materials as are especially adapted to such methods of transportation.

It is not even too much to predict that the future will see a great, deep-channel waterway, aside from the river itself—which is impossible of control as anything but a shallow stream—in the river valley and extending far inland to some great central seaport of the United States.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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RECONSTRUCTION OF THE NORFOLK AND WESTERN RAILWAY COMPANY'S BRIDGE OVER THE OHIO RIVER AT KENOVA, WEST VIRGINIA

Discussion.*

BY MESSRS. C. H. CARTLIDGE, T. KENNARD THOMSON, AND F. W. SKINNER.

Mr. Cartlidge.

C. H. CARTLIDGE,† M. AM. SOC. C. E. (by letter).—The authors have presented, in a clear and comprehensive manner, a description of an extremely difficult and interesting work, and are to be congratulated on the success attending their carefully worked out details. Such descriptions in minute detail are valuable to those having similar problems, and are not frequently available. For the authors' presentation of the processes of erection and their elucidation of the details required for the particular erection procedure the writer has sincere admiration.

There are some matters concerning general design which seem to the writer to be worthy of inquiry. The steel selected for the spans, having an ultimate strength of from 62 000 to 70 000 lb. per sq. in., is, if similar to that furnished for some other and larger work fabricated by the same company, an extremely reliable and valuable material, the writer having been privileged to assist in some interesting tests of large columns of such steel. It would seem that the assumed loading is large enough to provide for any requirement, and that the unit stresses allowed should be as large as safety would permit. The process of designing for the ultimate possible load is, it may be stated parenthetically, the only logical one, and greatly to be commended. It

* This discussion (of the paper by William G. Grove, Esq., and Henry Taylor, Assoc. M. Am. Soc. C. E., published in January, 1915, *Proceedings*, and presented at the meeting of February 17th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chicago, Ill.

Mr. Cartlidge.

is the practice of the writer in such cases. If it may be assumed that all the stresses which can come on the structure have been allowed for, it would seem that a material having a minimum yield point of, say, 34 000 lb. per sq. in., might be permitted a final or maximum working stress of, say, 80% of that, or about 27 000 lb. per sq. in. It is true that such working stresses are apt to produce somewhat of a shock to those who have so long been accustomed to thinking of stresses of 16 000 lb. per sq. in., but, with the postulates well in mind, the idea becomes reasonable. The unit stress of 24 000 lb. allowed by the designers for total stress, including, besides those due to live and dead loads, those due to wind and secondary stresses, seems somewhat conservative, and might well have been somewhat larger, say, 22 000 lb. for live, dead and wind load stresses, and not to exceed 26 000 lb. with secondary stresses.

The allowance for "dynamic effect" by the impact formula, $I = \frac{300}{300 + L}$, is the feature most open to criticism. For the chords of the long span, this gives an impact allowance of 36.6%, and in none of the many tests of long spans with which the writer is familiar, is there any evidence that such an impact effect is ever realized on the chord members of a span of this length. It is certain that for spans of 500 ft. or more, the impact effect of a train running at the speed which will impose a maximum dynamic effect on the span is not much, if any, in excess of 10%, and this does not vary widely between webs and chords. The writer is not aware whether it is expected to run trains over such long spans, built at such a great height above the water, at speeds sufficient to produce an impact on the floor stringers of about 91 per cent. It would seem doubtful. Of course, it is not within the province of fair criticism to charge the designers with having provided too strong a structure. It is the right of the owners to provide as strong a bridge as they are willing to pay for, but the use of this particular impact formula leads to the overloading of some members with unnecessary material, the result being that the bridge is not of equal strength throughout. The design of the large compression members might have been improved had the walls been, as far as possible, of single thick plates. It is doubtful if any quantity of stitch riveting of two or more plates will provide a compression member of efficiency equal to one having equal sections without rivets. It is the belief of the writer, also, that for sections as large as these, the **H**-section is worth all it costs in somewhat more expensive details.

Mr. Thomson.

T. Kennard Thomson,* M. Am. Soc. C. E.—This valuable paper is especially interesting to the speaker because he was Engineer of Bridges for the Ohio Extension of the Norfolk and Western Railroad,

* New York City.

having supervision of the old Kenova Bridge and some 129 smaller ones, in 1890-1901. Mr. Thomson.

William D. Janney, M. Am. Soc. C. E., of Baltimore, Md., was the Division Engineer at Kenova, and could add much interesting information relative to the foundations. These were all put in by the open coffer-dam method, there being practically no mud on top of the bed-rock, which made it very difficult to get a water-tight joint, especially as there was no really low water, to speak of, in 1890.

The records for years back indicated that the water could be expected to drop to 6 ft. every summer, and in times of flood to rise to 66 ft. Those who know the Ohio River realize how sudden the rises are.

The coffer-dams were very well built, of 12 by 12-in. timbers, forming a double wall all around the proposed pier, with a space of about 5 ft. between the two walls, which was filled with earth. Much trouble was caused by leaks, although many scow loads of material were dumped around the outside.

On the west side of the river a couple of coffer-dams had been placed in 1890, but not unwatered that season. Next spring, when the water in the river fell, the coffer-dams were found to be filled to the top—25 ft. deep—with sediment.

One of the coffer-dams on the east side of the river had given especial trouble from leaks, and an attempt was made to seal it by dumping 5 ft. of concrete under water. The concrete set well, except around the edge, where the water still leaked in as badly as before. The surface of the concrete was also found to be very uneven.

The erection of the steelwork was very rapid, as it was rushed on account of the danger of the falsework, 100 ft. high, being washed away. The dates for the erection, etc., are given in Table 3.

TABLE 3.—Record of Time of Erection, Etc., of the Old Kenova Bridge, During 1891.

	Span 1.	Span 2.	Span 3.	Span 4.	Span 5.
Length of span, in feet	301	304	521	304	301
Erection of falsework started	July 30	Aug. 15	Sept. 14	Oct. 23	Nov. 4
" " " stopped	.....	Aug. 17	.....	.....	.....
" " " recommenced	.....	Aug. 27	.....	.....	.....
" " " finished	Aug. 8	Sept. 6	Sept. 28	Oct. 30	Nov. 12
Erection of traveler started	Aug. 10	.....	Oct. 2	.....	.....
" " " finished	Aug. 13	.....	Oct. 6	.....	.....
Erection of ironwork started	Aug. 18	Sept. 8	Oct. 6	Oct. 31	Nov. 12
Ironwork finished and span swung	Aug. 25	Sept. 12	Oct. 21	Nov. 4	Nov. 16

Erecting a 300-ft. span in 3½ days was by no means a slow job. It was accomplished by the Baird Brothers, well known for their rapid work, and also for their recklessness.

Mr. Thomson.

In preparing for rapid erection, a little too much clearance was allowed between the stringers and floor-beams, with the result that, when the spans were swung free of the falsework, about thirty or forty of the connecting angles were cracked at the fillets and had to be cut out.

The four end piers were completed in 1890, and the three main river piers in 1891, the 100-ft. piers being built in less than 2 months each.

The actual costs of the substructures of this bridge were:

Masonry piers and abutments	$165 285
Viaduct pedestals	12 020
Shore foundations	6 932
River foundations	128 157
Inspection	4 288
Total cost	$316 682

The calculated weights of the superstructure were:

Viaduct	1 519 500 lb.
Four 298-ft. spans	2 734 000 "
One 518-ft. span	2 146 500 "
Total weight	6 400 000 lb.

Mr. Skinner.

F. W. Skinner,* M. Am. Soc. C. E.—Mr. Thomson's reference to the Baird Brothers brings reminiscences of those old-time bridge erectors who played a major part in the development of the art of American bridge building which has far surpassed that of all other nations. Bridge erection has not only kept pace with the design and fabrication of long-span bridges, but it has anticipated it, and by providing new methods, creating new apparatus, and training experts, has led the way and indicated the lines on which these great structures have progressed with a safety, speed, and economy unapproached abroad.

The art of bridge erection has been wholly and entirely created within the last 40 years, and some of its masters are yet winning great victories and establishing new records. When, less than 30 years ago, steel spans inaugurated the construction of trusses more than 550 ft. long, both plant and methods were simple and crude, and no special facilities existed, the multiple-drum hoisting engine was unknown, as was the pneumatic hammer, and very little power was available for field work, even on the largest jobs. Electric power was unheard of, and large hydraulic jacks were not on the market. Derricks and tackles seldom or never had capacities of more than 30 tons, wire-rope

* New York City.

tackle was rarely used, and single pieces or truss members did not exceed 30 or 40 tons weight and could not have been transported if they had. Mr. Skinner.

The erectors, however, never hesitated to swing long and lofty spans over torrent, chasm, and flood, and, working with much greater hardship and peril than now, succeeded in the face of great disadvantages. Many a time the erection was a race for life with ice, storm, flood, and scour, and the erectors were real men with splendid courage, loyalty, zeal, resourcefulness, and fidelity, ready for every emergency and anticipating the dangers and difficulties coolly, successfully, and modestly. Among those whom it has been the speaker's privilege to know best are some of the greatest of all, such men as William, Robert, Andrew, and John Baird, Robert Grimes, Milton Clay, the late H. F. Lofland, M. Am. Soc. C. E., H. A. Green, J. B. Gemberling, W. H. Wilkinson, John Watson, John Devin, L. N. Gross, Newt. Jarrett, Augustus Milliken, S. P. Mitchell, M. Am. Soc. C. E., Phelps Johnson, M. Am. Soc. C. E., and the engineers of the railroads and bridge companies. To them, to their comrades and disciples, some of whom are still directing the greatest constructions yet attempted, is due a large share of the credit for our most splendid bridges.

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CINDER CONCRETE FLOOR CONSTRUCTION BETWEEN STEEL BEAMS

Discussion.*

By Messrs. K. M. Boorman and T. Hugh Boorman.

Mr. K. M. Boorman.

K. M. Boorman,† Esq. (by letter).—The authors' conclusions as to fire resistance furnish added proof of the inadequacy of existing data on the proper thickness of fire protection of steel. The practice of specifying the placing of the reinforcement 1 in. above the exposed surface is based on an assumption which is not borne out by these investigations and others. A thickness of 2 in. is required as a protective coating for columns, and 1½ in. for beams. It is possible for such steel members, due to their comparatively large mass, to convey heat from their exposed surfaces quite readily. On the other hand, 1 in. only is required for the protection of slab reinforcement. This material, due to its small cross-section is, of course, more readily and to a greater extent affected by high temperatures. The specified 1 in. appears again inadequate when it is considered that in actual construction it is not possible, under economical working conditions, to insure this minimum thickness, and it is found to be reduced to ½ in. or less, and sometimes to nothing, that is, there is contact of the steel with the forms.

The writer is informed that the authors intend to extend their investigations, with a view to making recommendations as to the control of the quality of anthracite cinders. It might prove of value

* This discussion (of the paper by Harold Perrine and George E. Strehan, Juniors, Am. Soc. C. E., published in February, 1915, *Proceedings*, and presented at the meeting of March 3d, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. K. M. Boorman.

to determine the clinker-forming properties of different kinds and grades of coal, with a view of recommending a limitation of the use of cinders for concrete to the product of certain coals.

Mr. T. H. Boorman.

T. Hugh Boorman,* Esq.—This paper is of particular interest to the Engineering Profession as being in the line of more economical construction, which undoubtedly will be one of the principal features in the work of the Engineering Foundation. The utilization of waste material is of great importance. In America, with its immense natural resources, there has hardly been sufficient research on this question. The speaker, in his time, has seen the waste residua of gas-works and furnaces changed from useless and encumbering wastes to materials of commercial value, and hopes to see further developments with such an object in view.

In discussing the Report of the Special Committee on Materials for Road Construction, recently, the speaker had occasion to refer to Mr. Perrine's tests of one of the former waste materials, slag, in its connection with concrete construction, and would now suggest that he make tests and report on the utilization of another species of cinder, namely, the clinkers from garbage destructors or crematories.

In 1902 the speaker's attention was first attracted to the use of clinkers in concrete, in the City of Bristol, England, where they were utilized by Col. Yabbicum in the construction of cement sidewalks. On returning to New York the speaker reported to Mayor Low the advisability of building a destructor, and, after a lapse of some years, one was erected on Staten Island, in furtherance of that suggestion, but without consultation with the speaker. In 1911, in investigating the municipal plants of a number of English cities, the speaker again found how extensively these clinkers were utilized in concrete construction, and, in addition, found that they were used by the Borough of Kensington, London, England, in the manufacture of asphalt blocks. On an inspection of the crematory at Atlanta, Ga., in November, 1914, it was found that, through lack of appropriations, the clinkers could not be utilized as suggested, as old tin cans and other metal refuse were all baked together, and the resultant product could only be disposed of in filling land. The speaker has had no opportunity to examine the several destructors erected in the West, and therefore does not know whether their output could be utilized in the same way as in English cities.

A bill is now before the Governor of New York State for signature, authorizing the Borough of Manhattan to make a contract for the erection of a destructor to be operated by contractors. Such a project should be carried on by the municipality itself, for, though the speaker is not prepared to endorse a statement made that the receipts from

* New York City.

Mr. T. H. Boorman.

the handling and disposal of garbage by scientific methods, would pay the expenses of the Street Cleaning Department, he believes that a large revenue could be obtained therefrom and be devoted to paying such expenses. In view of this statement, it is hoped that the authors of this paper will include in their closing discussion a statement as to the comparative strength of concrete construction with clinkers, in comparison with other aggregates.

Should there be any difficulty in obtaining samples of properly coked clinkers in America, the speaker would be pleased to furnish samples from some of the English destructors.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOHN WYKEHAM JACOMB-HOOD, M. Am. Soc. C. E.*

DIED MARCH 6TH, 1914.

John Wykeham Jacomb-Hood was born at Lewisham, Kent, England, on January 12th, 1859. He was the second son of the late Robert Jacomb-Hood who, for many years, was the Engineer, and later a Director, of the London, Brighton, and South Coast Railway.

From 1870 to 1875, he attended Tonbridge School. He received his technical training at the School of Practical Engineering at the Crystal Palace.

In 1876, he was articled to the late William Jacomb-Hood, Chief Engineer of the London and South Western Railway, and from 1877 to 1887, was engaged in various offices of the Company on maintenance and construction.

In 1887, Mr. Jacomb-Hood was appointed District Engineer in charge of the London District, remaining in this position until 1896 when he was transferred, as Engineer, to the Western District with headquarters at Exeter, England.

In November, 1900, he was appointed Chief Resident Engineer of the London and South Western Railway, which position he held at the time of his death.

He died suddenly, from heart failure, on March 6th, 1914, at Dulverton, Somerset, England.

In his position as Chief Resident Engineer, Mr. Jacomb-Hood was responsible for many works of great magnitude, including the Woking to Basingstoke Widening; the extension of the road to Bulford Camp; the Bentley and Bordon Railway; the Axminster and Lyme Regis Railway; the new locomotive depot at Eastleigh; the introduction of power and automatic signaling on various parts of the System; the reconstruction of the bridges over the Thames at Richmond and Kingston; the construction of the stations at Salisbury, Basingstoke, and Clapham Junction; and the reconstruction of Waterloo Station which had not been completed.

Mr. Jacomb-Hood was a Member of the Institution of Civil Engineers and also of the Institution of Electrical Engineers. In 1911, he was President of the Permanent Way Institution. He is survived by his widow and a daughter, Miss Molly Jacomb-Hood.

* Memoir prepared by the Secretary from information on file at the Society House.

He was a man of marked personality and of such a genial nature that he will be greatly missed. As Chief Resident Engineer, he was always greatly interested in, and accessible to, the members of his staff and was greatly beloved by all who knew him.

Mr. Jacomb-Hood was elected a Member of the American Society of Civil Engineers on February 5th, 1902.

EMIL KUICHLING, M. Am. Soc. C. E.*

DIED NOVEMBER 9TH, 1914.

Emil Kuichling was born on January 20th, 1848, at Kehl, Germany. Owing to the failure of the Revolution of that year, his parents, Dr. Louis and Marie von Seeger Kuichling, moved in 1849 to Rochester, N. Y., where their son grew to manhood and spent the greater part of his life.

Mr. Kuichling received his early education in private schools. His taste for engineering showed itself when he was quite young. At the age of 14, he took the position of Chainman in the office of the City Surveyor of Rochester and in the following year became Rodman on the Western Division of the New York State Canals.

In order to complete his education, young Kuichling entered the University of Rochester in 1864, and, after taking the classical course, was graduated in 1868 with the degree of A. B. He stayed another year at this University, for a post-graduate course in engineering, on the completion of which he received the degree of C. E. While at the University his vacations were devoted to practical work in the Engineer Corps of the Western Division of the New York State Canals, and, after his graduation, he was appointed Assistant Engineer on this work.

Not satisfied with his theoretical education, Mr. Kuichling studied engineering, in 1870-73, at the Polytechnicum of Karlsruhe, Germany, at that time the most famous school of its kind in Europe, where he had Sternberg and Baumeister as instructors. After completing this course, he returned to the United States and, in 1873, was appointed Assistant Engineer on the Rochester Water-Works. In 1876, he was made Principal Assistant Engineer on these works, a position which he held until 1885, when he was elected a member of the Executive Board which had charge of the public works of Rochester. In 1887, he resigned from this office, in order to undertake the work of designing the sewerage system for the west side of Rochester. He was engaged in this work until 1890, when he was appointed Chief Engineer of the Rochester Water-Works, which position he held for ten years,

* Memoir prepared by Edward Wegmann, M. Am. Soc. C. E.

until it was abolished by a new City Charter which went into effect on January 1st, 1900.

In 1900-01, Mr. Kuichling was employed on the New York State Barge Canal as Engineer of Water Supply. In 1901 he moved to New York City, where he opened an office as Consulting Engineer, and had at once a large practice. While still in the various positions he held in Rochester, he was consulted about many other water-works, sewerage systems, etc. During the period from 1882 to 1892, he acted as Consulting Engineer to the New York State Board of Health. His advice was often sought in connection with hydraulic engineering works, and he was frequently employed as expert in important litigations.

Mr. Kuichling was married on January 28th, 1879, to Sarah Louise, daughter of John S. Caldwell, of Rochester, N. Y., who survives him.

He was a member of the American Water Works Association; the New England Water Works Association; the Public Health Association; the Rochester Engineering Society, of which he was the first President; the Rochester Academy of Science; the Delta Upsilon Fraternity and Phi Beta Kappa Society; all branches of the Masonic Fraternity; and the Engineers Club of New York.

Mr. Kuichling was one of the foremost hydraulic engineers in America. He was a great student and read and digested all the important articles, bearing on the specialty he had selected, which appeared in English, German, or French engineering literature. His memory was wonderful, and stored not only the interesting facts he had gathered, but the very places where they were mentioned. He was a born investigator, and probed to the bottom every subject he was called on to examine. An indefatigable worker, he rarely relaxed, and day after day, when engaged on some important piece of work, he would be at his desk or at his drafting-table until midnight. Above all, he was a man of absolute integrity and honor. In money matters, he was disinterested, almost to a fault. In dealing with subordinates, he had no patience with carelessness and negligence, but he had a kind heart and was always just.

In the death of Mr. Kuichling, the Profession has lost one of its brightest lights and a man of noble, sterling character. He always took an active part in discussing important engineering questions, and will be much missed at the meetings of the many technical societies to which he belonged.

Mr. Kuichling was elected a Member of the American Society of Civil Engineers on September 3d, 1884, and served as a Director in 1901-03, and as Vice-President in 1905-06.

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"COMPUTING RUN-OFF FROM RAINFALL AND OTHER PHYSICAL DATA." ADOLPH F. MEYER. (To be presented Apr. 21st, 1915.)

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OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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OF THE

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OF

CIVIL ENGINEERS

(INSTITUTED 1852)

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APRIL, 1915

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ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

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PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

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MINUTES OF MEETINGS

OF THE SOCIETY

March 17th, 1915.—The meeting was called to order at 8.30 P. M.; Director Arthur S. Tuttle in the chair; Chas. Warren Hunt, Secretary; and present, also, about 275 members and guests.

The Chairman announced that it had been arranged that this meeting be devoted to a general discussion of certain matters which it was intended to bring to the attention of the Constitutional Convention of the State of New York, to be convened April 15th, 1915; that the Board of Direction had appointed Messrs. Arthur S. Tuttle, Henry W. Hodge, and Alfred D. Flinn, to represent this Society at that Convention; that the Committee so formed had prepared certain suggestions; and that members of this Society, and of the American Society of Mechanical Engineers, the American Institute of Mining Engineers, and the American Institute of Electrical Engineers, had been invited to attend this meeting and take part in the discussion.

The Chairman further announced that the discussion was to be informal, and simply to enable the Committee to get the benefit of the opinion of engineers as to what changes in the Constitution of the State of New York would be advisable and expedient.

The general suggestion of the Committee was that it seemed desirable that the State Constitution, when amended, should insure:

"(*a*) A short ballot, thereby permitting close scrutiny of the qualifications of each candidate.

"(*b*) The selection of a professional head of a Department by appointment instead of election by popular vote.

"(*c*) An uninterrupted term of office for capable Bureau Heads, in order that an efficient government may be maintained.

"(*d*) The application of the Civil Service principle of selection for fitness to positions of major responsibility.

"(*e*) Giving the Chief Executive freedom in selecting his cabinet advisers, and placing upon him full responsibility for such selections."

A number of detailed suggestions concerning the office of the State Engineer and Surveyor; the appointment of Engineers; of a Department of Public Works, and a Department of Public Utilities, were presented by the Committee as subjects for discussion, and these suggestions were read by the Secretary, who also presented communications on the subject from Messrs. Clemens Herschel and Gano Dunn. A general discussion followed, which was participated in by Messrs. A. D. Flinn, William B. Landreth, T. Kennard Thomson, A. B. Fry, N. P. Lewis, E. W. Stern, H. Lichtenstein, T. Hugh Boorman, Henry B. Seaman, Ralph D. Mershon, T. H. Barnes, Oscar Lowinson, H. S. Harding, C. E. Conover, R. C. Kellogg, E. L. Corthell, B. H. Wait, J. H. Granbery, Kenneth Allen, F. W. Skinner, C. M. Hansen, B. C. Collier, E. J. Mehren, F. V. Henshaw, Chas. Warren Hunt, J. P. Carlin, O. H. Landreth, E. G. Carey, Allen Hazen, W. McClellan, R. S. Buck, S. C. Thompson, W. Goldsmith, G. R. Tuska, Bernt Berger, Henry W. Hodge, Lazarus White, and J. P. W. Richmond.

The Secretary announced the following deaths:

Chapman Love Johnson, of Casanova, Va., elected Member, October 7th, 1903; died March 11th, 1915.

William Edwin Moore, of Spokane, Wash., elected Member, February 7th, 1906; died January 24th, 1915.

William Gardner Russell, of Altura, Tex., elected Member, October 4th, 1905; died March 5th, 1915.

Adjourned 11.20 P. M.

April 7th, 1915.—The meeting was called to order at 8.30 P. M.; Director George W. Fuller in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 196 members and 39 guests.

The minutes of the meetings of February 17th and March 3d, 1915, were approved as printed in *Proceedings* for March, 1915.

A paper entitled "The St. John Levee and Drainage District of Missouri", by R. M. Strohl, Jun. Am. Soc. C. E., was presented by title, and discussions by Messrs. Vernon M. Eager and J. S. Spiker were read by the Assistant Secretary.

T. L. Condron, M. Am. Soc. C. E., addressed the meeting on "The Effects of Fire on the Reinforced Concrete Buildings of the Edison Plant at West Orange, N. J., and the Methods of Making Repairs", illustrating his remarks with lantern slides. The subject was discussed by Messrs. H. I. Moyer, J. P. J. Williams, W. J. Barney, E. W. Stern, Carleton Greene, and T. L. Condron.

A vote of thanks was tendered to Mr. Condron for his very interesting address.

The Assistant Secretary announced the election of the following candidates on April 7th, 1915:

As Members

Reginald Sydney Courtney, New York City
Francis Hewette Davis, San Francisco, Cal.
Edgar Raymond Frisby, Manila, Philippine Islands
Thomas Nixon Gilmore, New York City
Harry Andrew Hageman, Boston, Mass.
Robert Lacy, Baltimore, Md.
Ferdinand Joseph Litter, New York City
Phillips Bathurst Motley, Montreal, Que., Canada
Seth Augustine Moulton, Portland, Me.
George Harvey Norton, Buffalo, N. Y.
Zenas Newton Vaughn, Boise, Idaho
Frederick William Whiteside, Denver, Colo.

As Associate Members

Albert Hilands Acher, Balboa Heights, Canal Zone, Panama
Elmer Ellsworth Adams, Seattle, Wash.
James C Bell, Schenectady, N. Y.
Charles Benham, Jr., Rochester, N. Y.
Hjalmar Luther Blomshield, Trenton, Mich.
Clifford Holmes Boardman, Oakland, Cal.
Allen Roy Boudinot, Davenport, Iowa
Richard Brunel, Portland, Me.
Louis Schumann Bruner, Montreal, Que., Canada
Percy Hiram Budd, Phœnix, N. Y.
Robert Lee Burney, San Antonio, Tex.
Robert Campbell, Detroit, Mich.
Clifford Earl Chase, Brewster, Wash.
Charles Henry Covey, Montreal, Que., Canada

George Dave Curtis, Tallahassee, Fla.
Thomas Charles Desmond, New York City
Joseph Alfred Elliott, Wheatland, Wyo.
Frederic Butterfield Faitoute, New York City
Joseph John Greene, Sydney, Australia
James Birney Griffin, Venice, Cal.
Arthur Lines Harris, Monte Christi, Dominican Republic
Elmer Perkins Haw, Ancon, Canal Zone, Panama
Samuel Darlington Heed, New York City
Whitney Clark Huntington, Boulder, Colo.
Ralph Hutchins, Baltimore, Md.
Clarence Cecil Jacob, Phœnix, Ariz.
Cleveland Abbe James, Buffalo, N. Y.
Robert Waterman Jones, McMinnville, Ore.
Warren Winfield Kelly, Los Angeles, Cal.
Frederick Evoy Kneip, Washington, D. C.
Gerald Wilson Knight, Lakewood, Ohio
Jean Howard Knox, Oakland, Cal.
Louis Dietrich Koop, New York City
Clyde Emerson Learned, Oswego, N. Y.
Clifford Stone Leet, Pittsburgh, Pa.
Albert Richard Losh, Manhattan, Kans.
Lee Orlo Murphy, Reno, Nev.
Ray Stevens Quick, Pittsburgh, Pa.
Frank Oliver Ray, Denver, Colo.
Lawrence Dunlap Sheppard, Keokuk, Iowa
James Raymond Stack, Duluth, Minn.
Norman Merritt Stineman, Chicago, Ill.
Nathan Henry Sturdy, Youngstown, Ohio
Frank Henry Villie, South Bethlehem, Pa.
Willis Dow Peck Warren, Decatur, Ill.
Ernest Clifford Willard, Seattle, Wash.

As Juniors

Dumont Beerbower, Washington, D. C.
Pablo Tomás Beola, Manati, Cuba
John Wolfram Berkefeld, Piedmont, Cal.
Donald Hilary Connolly, Fort Sam Houston, Tex.
Elwin Harold Findlay, Bridgeport, Conn.
Arthur Sidney Hallberg, Manila, Philippine Islands
Fred Dailey Hartford, Denver, Colo.
Henry Jalmar Heinonen, New York City
Frank Helm, La Junta, Colo.
George McIntyre Holroyd, Albany, N. Y.
Charles Henry Elliott Holstlaw, West Palm Beach, Fla.
David Howe Lucchesi, Jr., Baltimore, Md.

Walter Scott Obermeyer, Pittsburgh, Pa.
Norris Dunham Pease, West Hartford, Conn.
Clarence Paulding Rhynus, Washington, D. C.
Joseph Joslin Strachan, Savannah, Ga.
Carlos Nicolás Todd, Havana, Cuba
George Boynton Watson, Grand Mere, Que., Canada
Roy Allert White, Louisville, Ky.
Ralph Edward Whitney, Lynn, Mass.

The Assistant Secretary announced the transfer of the following candidates on April 7th, 1915:

From Junior to Associate Member

Paul Bailey, Tracy, Cal.
Harold Affleck Brainerd, Westfield, N. J.
Lynn Crandall, Salt Lake City, Utah
Royal William Davenport, Washington, D. C.
Marcus Martin Farley, New York City
John George Hirsch, Milwaukee, Wis.
Arthur Caswell King, Springfield, Mass.
Jabez Curry Nelson, New York City
Albert Fredrick Porzelius, Pittsburgh, Pa.
Earl Querbach, Avalon, Pa.
Richard Francis Roberts, Nutley, N. J.
Shaler Gordon Smith, Rockford, Ill.
David Bernard Steinman, New York City
Otto Jordan Swensson, Yonkers, N. Y.
Gordon Saxton Thompson, Troy, N. Y.
George Lansing Wentworth, Yonkers, N. Y.

The Assistant Secretary announced the following deaths:

Foster Crowell, of New York City, elected Member, December 1st, 1880; died March 29th, 1915.

Francis Henry Hambleton, of Baltimore, Md., elected Member, March 5th, 1873; died March 19th, 1915.

William Hunter, of Philadelphia, Pa., elected Member, June 5th, 1895; died April 2d, 1915.

Walter Ashfield McFarland, of Washington, D. C., elected Member, May 3d, 1910; died March 17th, 1915.

Henry Gurney Morris, of Philadelphia, Pa., elected Member, December 4th, 1867; date of death unknown.

Isaac Rich, of Somerville, Mass., elected Member, May 6th, 1903; died March 11th, 1915.

Adjourned.

REPORT IN FULL OF THE MEETINGS OF THE PRESIDENTS OF LOCAL ASSOCIATIONS OF MEMBERS, AMERICAN SOCIETY OF CIVIL ENGINEERS.

A meeting of the Presidents of Local Associations of Members of the American Society of Civil Engineers was held at the Society house at 10.30 A. M., January 19th, 1915, upon the invitation of the Board of Direction.

Present: A. M. Hunt, President, San Francisco Association; E. F. Vincent, President, Colorado Association; J. L. Hall, Delegate, Seattle Association; John B. Hawley, President, Texas Association; W. B. Gregory, Vice-President, Louisiana Association; F. W. Cappelen, President, Northwestern Association; U. B. Hough, President, Spokane Association; F. G. Jonah, Vice-President, St. Louis Association; C. T. Leeds, President, Southern California Association; Willard Beahan, President, Cleveland Association; Leonard Lundgren, Representative, Portland, Ore., Association; J. E. Greiner, President, Baltimore Association; Park A. Dallis, President, Atlanta Association; Richard L. Humphrey, President, Philadelphia Association.

Also present: Hunter McDonald, President, Am. Soc. C. E., and a Committee of the Board of Direction appointed by resolution to attend this meeting, consisting of J. A. Ockerson, M. T. Endicott, Past-Presidents, and Charles Warren Hunt, Secretary.

On motion, duly seconded, Richard L. Humphrey was elected Chairman.

THE CHAIRMAN.—I might say just a word in acknowledgment of the compliment you have paid me. I think these local associations can be made a very important factor in the work of the Society, and I certainly feel highly honored that I should be selected, at this preliminary meeting, to act as Chairman.

Perhaps it would be best to start by having our Secretary outline briefly the general scope of this meeting. I understand that a committee of the Board of Direction is to meet with us, and I think the Secretary might explain the purposes of this meeting, and then we can take up the work to be done.

CHARLES WARREN HUNT, SECRETARY, AM. SOC. C. E.—Mr. Chairman and Gentlemen, in the first place, the Committee of the Board consists of Messrs. Ockerson, Endicott, and Swain, and the Secretary. The Committee was not appointed to interfere with the business of this meeting, but it was thought that it might be helpful to the Presidents of the local associations to have some members of the Board present during their deliberations.

As to the question of the work of this meeting, I think no one knows just what should be done. These associations are widely scattered, and different as regards their make-up and as to local condi-

tions. Some of them are in large and flourishing cities, and some are in smaller places, and the Board of Direction does not know the ideas of the members of these various associations in respect to their relation to the Society. It has never had a chance to find out, and this meeting has been called so that there might be a general discussion of the conditions that obtain and the needs of the local associations, with a view to outlining the things which it is wise and proper for such associations to undertake, local or otherwise; how far their powers should go, and any question which might come up as to the relations of the associations; further than that, nothing has been talked of, except in a very general way. It was thought that if the representatives of these associations could get together, much good might result by outlining a general plan for uniformity of action in their work.

JOHN A. OCKERSON, PAST-PRESIDENT, AM. SOC. C. E.*—Mr. Secretary, could you give us the total number of members in the fourteen associations?

MR. CHARLES WARREN HUNT.—I could not; I should think the San Francisco Association is the largest of them.

A. M. HUNT, M. AM. SOC. C. E.—188.

MR. CHARLES WARREN HUNT.—And Seattle, Portland, and Spokane —there are about 100 members in those associations.

J. L. HALL, M. AM. SOC. C. E.—About 80 in Seattle.

U. B. HOUGH, M. AM. SOC. C. E.—There are 30 in Spokane.

W. B. GREGORY, M. AM. SOC. C. E.—35 in New Orleans.

JOHN B. HAWLEY, M. AM. SOC. C. E.—68 in Texas.

MR. CHARLES WARREN HUNT.—You take in the whole State?

MR. HAWLEY.—Yes. In the State Association we have 142 on the list, and 68 are members of the association.

MR. CHARLES WARREN HUNT.—Why can't we call the roll?

THE CHAIRMAN.—I suggest that we elect a Secretary, and following that, that each representative shall state the number of members in his section, the territory covered, and the work performed. The Chair will entertain a motion to nominate a Secretary.

A MEMBER.—I nominate Mr. Park A. Dallis, of Atlanta, to be Secretary.

(Motion seconded.)

THE CHAIRMAN.—Any further nominations? Those in favor of Mr. Dallis as Secretary will signify by saying "aye"; contrary minded, "no." It is agreed to.

* On February 11th, 1915, proofs of this Report were forwarded to all who took part in the meetings, with the request that they be corrected and returned promptly. All have been heard from except Mr. Ockerson. The record of his part in the meeting is here printed as taken down by the reporter.—*Secretary.*

Inasmuch as San Francisco was the first association started, Mr. A. M. Hunt might perhaps outline briefly the membership, the territory covered, and the scope of the work performed by the San Francisco Association.

Mr. A. M. Hunt.—The San Francisco Association was the first one organized (on April 28th, 1905), and it embraces those members of the Society resident in San Francisco, and, I think, within a radius of 100 miles, although of that I am not positive; the present membership is 188.

We have meetings six times a year, and they are generally held at one of the hotels, preceded by a dinner, with a short intermission, followed by papers, or the presentation of some topic of general or local interest, usually accompanied by lantern slides. The meetings have been very active and profitable. The average attendance during the last year was about 70. Since its organization, the association has imposed local dues of $5, and the fund thus created is used for expenses of meetings, such as for lanterns, etc.; also, the President and Secretary are authorized to invite, as guests of the association, at these dinners and meetings, any visiting members of the Society; and at almost all the meetings we have two, three, or four visiting members.

Quite a little fund has accumulated, and it is our hope that eventually that fund may grow to such a sum that we may use it for some really creditable purposes. I think the San Francisco Association, as well as all others, has felt the need of something definite as to its powers and limitations. The question has frequently come up as to action on matters of general and public interest. We do not know where we stand, except that, in all matters of national or country-wide interest, we feel that the local associations should be very careful as to any action which they may take, which might be considered as committing the Society, and I think something definite should be outlined which would state the limitations as to what we may or may not do, and the reasons for it.

However, whenever matters of wide importance come up and action is taken by the Board of Direction of the parent Society, the local associations should be informed fully and constantly as to that action, and what action seems desirable from the point of view of the Board of Direction, so that the local associations may reinforce and strengthen the influence of the Society at large. It would be fatal—for instance, take one subject which is now active, that of licensing engineers—if the local association in one part of the country should take action which differed from that taken by the parent Society, or by associations in other parts of the country. It would be quoted against us, and would also be quoted as the action of the entire Society. That would be very undesirable.

On the Pacific Coast I think we have as nearly a class of insurgents as anywhere in the country, and although we desire as full and complete local autonomy as we can get, we feel that there are things which we have no right to trench upon.

The local association should be given full power to act on all matters which are of local interest only, or where the action taken by the local association could not be quoted or used in other sections of the country.

Questions have frequently arisen where the association could have helped mould public opinion, and perhaps rendered public service, but we have hesitated to take action—to take steps as an association—because we did not know how it would be regarded by the governing board of the Society. It would be easy to outline general rules which would govern in these matters; and it is one of the most important things for this meeting to consider.

The Chairman.—I think we will take the gentlemen in the order in which they sit.

Secretary Dallis.—I would like to ask Mr. Hunt if his association collects dues from the total membership?

Mr. A. M. Hunt.—Dues are collected only from members of the association. Any member of the Society who expresses a desire to join may do so, and only those who join as members are assessable for dues.

A Member.—You collect dues from 188 men?

Mr. A. M. Hunt.—Yes.

The Chairman.—Will you state the territory that you cover?

Mr. A. M. Hunt.—I cannot give it off hand.

Mr. Charles Warren Hunt.—The State of California.

Mr. A. M. Hunt.—The entire State of California?

A Member.—There is a Southern California Association.

Mr. A. M. Hunt.—Our title is the San Francisco Association of Members of the American Society of Civil Engineers.

The Chairman.—It would be well to take the gentlemen in order: Mr. John B. Hawley, President of the Texas Association. Will you state the number of your members, the dues, if any, the territory you cover, and the scope of your work?

Mr. Hawley.—We cover the entire State of Texas. The catalogue shows 142 members throughout the State. If we meet them, we remind them that they should be members. Our dues are $3 a year. Our Constitution and by-laws were copied almost exactly from those of the San Francisco Association, only a few details being changed. We meet twice a year, in May and November, at some point in Texas. The first meeting was held on November 13th, 1913, in Dallas, and about 35 or 40 members attended. We organized and had a few discussions—no formal papers—and a dinner, and left to the Board of Directors the matter of locating the next meeting.

The next meeting was held at Austin, under the auspices, you might say, of the State University. At that meeting several papers were read, and the Secretary had them printed. They were corrected by the authors, and printed in a small volume of *Proceedings*. The Directors located the first annual meeting at Houston, Tex., where there was an attendance of about 65, and several papers. Our *Proceedings* are not yet printed. I tried to get them printed so that I might bring them with me.

The *Proceedings* will be of the standard size and the papers are revised by the Board of Directors before they are presented. We had several quite creditable papers. Our next annual meeting will be in San Antonio. These meetings have been in South Texas, because the majority of our membership is south of the central line, and east of the central meridian of the State. In May or the latter part of April we meet, at the time of a fête held there, usually in the latter part of April.

The next meeting will be in Fort Worth. We are distributed over 300 000 sq. miles of territory, but we aim to make of the social feature quite an element. Some of us have been in Texas for 20 years. I have been a member and located at Fort Worth more than 20 years, and yet I do not think I have met all the members in that section.

The matter of getting together and becoming better acquainted is of importance; but we are not overlooking the matter of getting more and more a set of scholarly papers presented to the meetings. We have very good engineers among our members, and there is no reason why we could not get them to write papers, and print them in our own *Proceedings,* and send them to the Society for filing, and that will be done.

THE CHAIRMAN.—The next gentleman, Mr. Gregory.

MR. GREGORY.—I am Vice-President of the Louisiana Association, at New Orleans. The President of the Association, Major Frank M. Kerr, found it impossible to come to this meeting, and therefore asked me to represent him. Our association has about 35 members, of all grades, and the total membership of the Society in Louisiana is between 55 and 65. Not all the members, therefore, have come into our local association. We organized as a result of the meeting of the American Society of Civil Engineers, held in New Orleans, in October, 1913.

The active work of the association has only begun during the last few months. Last Wednesday night we had a meeting which was followed by a dinner, the annual banquet, and at that time we discussed a paper which I am sure you will all remember, one by Maurice G. Parsons, Jun. Am. Soc. C. E., entitled "The Philosophy of Engineering".

The meeting was quite well attended, and very much enjoyed. We had had one meeting, three months previous to that time. We have only four meetings a year. The reason is that the Louisiana Engineering Society was organized in 1898, and was a strong, going concern before the local association of the American Society of Civil Engineers was organized, and the Louisiana Engineering Society embraces mechanical, electrical, and all other lines of engineering, and covers a wider field. Practically every member of the local association is also a member of the Louisiana Engineering Society, and we have felt that whatever we do must be done so that we will not interfere with the workings of the latter society.

I came here without any instructions, merely to listen and to learn, and to aid in any way possible in any movement that will lead to the betterment of local societies, or of the American Society of Civil Engineers, and add any interest to the work.

THE CHAIRMAN.—The next gentleman.

F. W. CAPPELEN, M. AM. SOC. C. E.—The Northwestern Association was organized in the fall of 1914. We have had two meetings. I do not know how many members we have, because we take in North and South Dakota, Minnesota, and part of Wisconsin, and the Upper Peninsula. Our Secretary has been corresponding with various members, but I really do not know how many we have, but we have had two meetings with about 40 members present, mostly from the greater cities, some from South Dakota, and also members from Michigan. We adopted practically the San Francisco Constitution. Our dues are $2.

There is one thing that I might mention. Minnesota belongs to the tail end of Geographical District No. 3, which takes in New York State, and we have very little chance for representation. We would like the possibility of changing the district, to get away from the tail end of New York State. We are so far away, and, of course, we have absolutely nothing to say in the affairs of the district. If we could make a change in that line, we would like it very much.

MR. OCKERSON.—Why don't you make New York the tail end of your district?

MR. CAPPELEN.—Maybe we can; I had not thought of that. The meetings have been very well attended, and I believe it is a very good move. I am here to learn and listen.

MR. CHARLES WARREN HUNT.—I would like to ask why it was that you took in so much territory, and whether it was thought that people would come to meetings from Montana and other outlying points, and that there would be a possibility of their really belonging to the local association.

MR. CAPPELEN.—The matter was taken up by correspondence, and by committee.

Mr. Charles Warren Hunt.—And the people out there really wanted it?

Mr. Cappelen.—Yes, sir.

Mr. Charles Warren Hunt.—I think that is a good point.

Mr. Cappelen.—We have the Twin Cities as headquarters, and meet at the Engineering Building of the University of Minnesota, Minneapolis, where we have splendid facilities, an assembly hall as large as the Auditorium of the Society House, and good facilities for consultation. We have a caterer, and have dinner served in that hall as well as you could get it in a first-class restaurant. We open our meetings with a dinner, and have a couple of hours of discussion of some subject.

Mr. Charles Warren Hunt.—My object in asking was to bring out the idea that some of these associations have confined themselves to State lines, but yours has not, because there are a lot of outlying people in it who are not in touch with any particular community, and not as much in touch with things in general as those who live in the smaller cities, and it might be proper for this meeting to consider the enlargement of the lines of the various districts, so as to take in the membership which is not represented, in order that, as far as possible, every member of the Society could belong to a local organization.

Mr. Cappelen.—We have a member from the copper country, one from Marquette, and one from Pierre, S. Dak. They have been present twice, coming on quite long trips. We made a special effort to get these gentlemen to come to us, because we believed that most of these engineers would be glad to meet the Twin City engineers, and join this association. We have taken in the new banking district, what do you call it?

Mr. Charles Warren Hunt.—The Federal Reserve.

Mr. Cappelen.—Yes, we take in the territory of the Ninth District of the Federal Regional Bank.

Mr. Ockerson.—There is an amendment up now that will change these districts.

Mr. Charles Warren Hunt.—Yes, I did not mean districts, but there are many members who are in the outlying country, and it would be very advantageous if they could be attached to one of these local associations.

The Chairman.—Mr. Cappelen referred to the fact that his association is in the same district as New York State, although it is away off in the western part of the country. There is an amendment pending that will make thirteen districts.

Mr. Cappelen.—You know Minnesota belongs to the New York district.

Mr. Charles Warren Hunt.—Yes, that is the weakness of the present system. It is certainly the intention of the Board of Direc-

tion in proposing this amendment to the Constitution to provide six more districts, and that will enable the territory to be divided in such a manner that there would be at least one, and possibly more associations in each district, so that every member of the Society would have the privilege of joining one of these local associations.

Mr. Cappelen.—We feel that we should get a little better representation on the Nominating Committees, and so forth. Generally, it is this way in Minnesota: We get a letter from some candidate, saying, "will you help us", and some time after we get another from somebody else. We have nothing to say; we do not know anything about it. We are so far away that we have really no interest whatever, nor any influence.

Mr. Charles Warren Hunt.—You are not much worse off than some of the other districts. You see, Seattle and Denver are in the same district, and Texas, I think, is in the same district as Washington, D. C.

Mr. Cappelen.—We belong to New York State, you see.

The Chairman.—Mr. Hough.

Mr. Hough.—I am the President of the Spokane Association. Our membership is 30; I believe that is what the Secretary and I counted the other day. For some little time it was around 25, but some new members have come in lately, and the work now in progress by the various railroads will make our membership more, and our association, I think, larger. Our dues are $2, and we meet every 60 days. We usually have a paper, or at least a discussion, on some engineering subject. Our association is rather small, but we felt that there was certain work that we ought to do, we did not know whether we ought to take it up through the American Society or not. The result was that we have just organized, and gotten into working shape, a General Engineering Society. We organized the local branch of the American Society of Civil Engineers about three years ago. However, I was not in the State at the time. A great deal of my work is outside the State, and I am absent much of the time, but we feel that we ought to do a great deal of work that is now being done by other people. In other words, the engineers of Washington are not getting through the State what they are entitled to, so we have organized a General Engineering Society to take up that work, as we thought we could do better in that way. We have a membership of 150 or 200 in the new organization. That covers the Electricals, the Mechanicals, and the Civils; it takes in the Architects, and I think the Chemists. I know we intended to cover all those branches. We are taking up the Water Code of the State, in fact, the American Society of Civil Engineers took it up two years ago. Another matter we are taking up, through the Society, is the road work in the State, which is being done by politicians. We feel that the engineers ought

to get in on this. The idea of organizing this new society was to get after this work, but we did not know whether we ought to do all these things as members of the American Society of Civil Engineers or not.

I hope that the work of this Society will be enlarged. There is a Society of Civil Engineers at Seattle, I believe it is Northwest or Northwestern——

Mr. Charles Warren Hunt.—The Pacific Northwest Society.

Mr. Hough.—That is it. They tried very hard to start a Pacific Northwest Society of C. E. in Spokane about three years ago, but we felt that we wanted some kind of a yard-stick to measure up the members which we did not believe this society had; so we got together and organized the Local Association of the American Society. The Electric Society was organized at the same time, and both of them, though small, are in a very flourishing condition.

Secretary Dallis.—I want to know what the total membership of this Engineering Society is.

Mr. Hough.—Thirty, I said.

Secretary Dallis.—That is the Association; how many are there in the Spokane Engineering Society?

Mr. Hough.—The other Society?

Secretary Dallis.—Yes.

Mr. Hough.—200.

F. G. Jonah, M. Am. Soc. C. E.—I am Second Vice-President of the St. Louis Association. The President of that Association, Mr. Ockerson, is present, and can tell you more about it than I can.

Mr. Ockerson.—As President of the Association, I appointed myself delegate to this meeting, but President McDonald informed me that that would not answer the purpose, that I would not be recognized as a delegate; so I appointed Mr. Jonah, who, I assumed, would tell all about our association. The members of the St. Louis Society of Civil Engineers have held out for a long time against any formal association, because St. Louis has an active Engineers' Club, and has had for 40 years or more, and the members did not feel that they could afford to do anything that would interfere with the efficiency of their club. They, however, had an informal association which met at irregular intervals. Whenever anything of importance in connection with this Society came up, a meeting would be called by the President, and those matters discussed, and some action determined upon; but beyond that we had no regular affiliation with the parent Society. The St. Louis Engineers' Club, within the last two years, has adopted a different scheme, and now we have an Associated Engineering Society, the Engineers' Club being at the bottom of it. Each brings in the members of all the other societies, and makes really a very active organization, and very effective, the Electricals, the Mechanicals, and the Mining

and Contracting Engineers, and all have a certain interest in the proceedings of the Engineers' Club.

During the past year the members of the American Society of Civil Engineers concluded to organize more formally, and adopted a Constitution which was submitted to and approved by the Board of Direction of this Society. We have no definite meeting dates, except that two meetings a year are called. Still, whenever there is anything of importance relating to the welfare of the Society, a meeting is called for the consideration of such subjects as come up.

We have participated with the Engineers' Club; two meetings a year are assigned to the American Society of Civil Engineers, and members of the Society are expected to provide the papers, and a member of the local association presides at these meetings.

We have on our list, I think, about 180 members. However, there are only about 65 who are paying members at the present time. Our dues are fixed at $2 a year. I think that about covers the ground. We feel that the association can do a great deal of good in the way of sustaining the Society, and can increase its scope of usefulness and influence by taking some assertive action in public matters in which the Society should participate.

Some of our members feel that they ought to have the same privileges as members of the other societies, the members of the Mechanical Engineers, for instance, and they ought to be allowed a certain sum per member on account of being members of the local association. There is quite a difference of opinion in regard to that, however, and I think most of the members feel that the dues paid to the Society are fully compensated by the value of the membership and the *Proceedings* and matters of that kind. I have paid dues for 35 years, and I can say that I think it is the best investment that I have ever made.

Mr. Charles Warren Hunt.—Colonel Ockerson therefore does not have to pay dues any more.

Mr. Ockerson.—Then, Mr. Secretary, I cannot get any rebate. I do not know that there is anything that I can add. As I say, we are tied up with the Local Engineers' Club to such an extent that we do not feel that we ought to do anything that will interfere with its activities. We have concluded to take in a radius of 200 miles from St. Louis. That is, we do not want to trench on Chicago and take in their members—that would not be fair—or on Kansas City, on the other side, and other cities not far away. So we have limited our activities to a radius of 200 miles from St. Louis.

The Chairman.—Did you say that the members of the Society voluntarily become members of the association?

Mr. Ockerson.—Yes.

Mr. Charles Warren Hunt.—There are about 65 paying members.

THE CHAIRMAN.—Yes; but they are not members until they subscribe.

MR. OCKERSON.—We hold them as members for the time being, but we have to contribute to the Engineers' Club in proportion to our paying members. Consequently, although our circulars are sent to all, we limit our active members to those who pay, because we are obliged to pay the Engineers' Club a certain sum per member.

C. T. LEEDS, M. AM. SOC. C. E.—The San Francisco Association covers the territory of the Southern California Association, but the members in Southern California felt that the distance to San Francisco was so great that there was need of an association there. There has been for some years in Los Angeles what is known as the Engineers' and Architects' Association, which has been quite a useful association, and includes engineers of all branches, and also architects, but there was a feeling among a good many men, that, if we had a local association, as members of the American Society of Civil Engineers, there was a great field for us. So our association was formed in the fall of 1913, with a charter membership of 85. Since then we have added eight members, and there remain approximately 50 members of the American Society of Civil Engineers who have not yet joined, but whom we are trying to persuade to do so.

MR. CHARLES WARREN HUNT.—What is the limiting line on the north of your territory?

MR. LEEDS.—I am not certain whether it is stated in the Constitution or not, but I think it is the topographical division.

MR. CHARLES WARREN HUNT.—I did not know whether it was or not.

MR. LEEDS.—Yes, I think it is that. We copied our Constitution directly from the San Francisco Constitution, making the membership dues $3 instead of $5, and holding our meetings bi-monthly.

We have had a very good attendance at each of the meetings, where, as is customary, we have a dinner, and a paper afterward. Another activity which we have, which we think will serve greatly, is that of holding informal weekly lunches, simply of the men whom we can get together informally at some convenient restaurant—no papers, but simply to come together for social intercourse, and to discuss matters of engineering interest.

There are two matters in particular which we are agitating at the present time, in the endeavor to make the stand of engineers higher, one in particular, on expert testimony. I am not familiar enough with the laws in the East, to know much about the situation here, but out there it is a fact that certain engineers have acquired the reputation of being professional experts; in other words, their services are available to the party who is willing to pay them the highest. Fortunately, the doctors have in their field much the same conditions, so that there has

been, or is to be, introduced at this session of the State Legislature a bill which will require that an expert, either a doctor or an engineer, shall be appointed by the Court, and shall simply testify as to facts, and in that way his testimony will not be salable.

Another matter which has been taken up is the need of more data as to rainfall and snowfall in the mountains. Doubtless, Mr. Hunt, of the San Francisco Association, has had experience of this kind, which I think is general throughout the Pacific Coast, in investigating questions of power installations, and matters of that sort.

In Los Angeles, in connection with flood control, there is an astonishing lack of information at the rainfall stations, where there should be much. A report was presented by Mr. Binckley and Mr. Lee, before one of the meetings of our association, and has been forwarded to the Board of Direction for revision and possible publication.

Mr. Charles Warren Hunt.—I would say, Mr. Chairman, that this report is endorsed by the Southern California Association, has been accepted, and will be published as soon as possible.

Mr. Leeds.—I am very glad to hear it.

Mr. Charles Warren Hunt.—The Board accepted it on the statement that it had been endorsed by the Southern California Association; it is going out as such; the Southern California Association is accepting responsibility as to all the statements made.

Mr. Leeds.—The association is perfectly willing to do that.

Mr. Charles Warren Hunt.—That was the feeling of the Board, that if any association sends in any paper and stands sponsor for it, it is proper for the Society to publish it.

Mr. Leeds.—We do not want to commit the Society in any way.

Another matter which has been discussed very informally by us is that of the admission of members to the parent Society. I suppose it is common throughout the country, but we have run across it particularly out there. A man wants to join the Society, and he will either approach a member personally, or write to him, and ask if he has any objection to putting his name down as an endorser. It is rather a delicate thing to turn a man down, and at the same time if one is not careful about it, the name goes down, and the impression goes out to the other members that that name has been endorsed; so that we would suggest that wherever an application is made, regardless of the endorsements which may be received from individuals, a letter be addressed to the local association requesting its Board of Direction, or a committee which it might appoint, or a vote of the entire association, whichever might be deemed best, as to their recommendation regarding the admission of the applicant and the grade to which he is eligible.

If it is found best, the parent Society need not delegate any power, the local association may not be given power to blackball a certain man, or to guarantee his admission; but it should give additional light to

the Board of Direction, and where concerted action of that sort is taken, it certainly should outweigh any other recommendation.

Mr. Ockerson.—The matter of expert testimony came up among the St. Louis members a few days ago, and it seems to me that if the Engineering Societies can take a hand in a movement of that sort, which will result in eliminating the kind of expert testimony we have, it would be the best thing we could do. I have conferred with a number of the men of the local associations, and they say that they do not seem to be able to reach any conclusion. If it could be taken up at the expert end of the line, we might reach some conclusion.

I once drew a specification for a Board in power, which did not suit the successors of the Board, who were of a different political complexion. It was suggested to me that the next time I drew a specification I should have it mean the same thing to any politician. I found it impossible to do that.

This is very important, and if our Society could take the lead in matters of this kind and get some results, it would be very desirable.

Mr. Leeds.—Then there is a question of privilege. In a case that arose very recently, one of our members was in Court as an expert, and had rendered a report. The other side learned of it, and nothing could prevent that report from being brought into evidence. It is inadmissible to make the communications of an engineer a privileged matter.

Mr. Hall.—I am a delegate from the Seattle Association. Mr. Ernest B. Hussey, the President, was unable to attend, and asked me to come in his stead. On my way here I prepared a little paper which contains some recommendations to submit to this body. I will present them now, or reserve them until later.

The Chairman.—Each association should present its views in a general way, and now is as opportune a time as any.

Mr. Hall.—(Reading.)

"I desire to offer a few remarks upon the relations between local sections and the parent Society and to suggest some ways in which it is thought a local section may become most useful to the Society, to its own members, and to the community in which it is situated. Incidentally, I will make brief mention of the Seattle Association of members as illustrating one phase of the subject.

"The want for local divisions of the American Society of Civil Engineers is clearly evidenced by the voluntary association of members already formed, and they in a measure automatically supply that want.

"In order that the local sections may become fully effective it is felt that provision for them should be made in the organic law of the Society, and that they should be accorded representation and a voice in its general councils and conventions. Also, the relations should be so adjusted that local sections will tend to embrace the entire membership.

"The surest way to induce every member of this Society to become a member of a local section will be to enable him to do so without extra expense to himself. Such an arrangement is open to the objection that it will deprive the general treasury of a certain amount of revenue which is required to meet the expenses of local sections. Our experience shows, however, that this necessary expense is comparatively small.

"If each local section be authorized to collect from its own members the annual membership dues and remit to the Society, after retaining, say, $2 per member to meet local expenses, then the local section will have a strong incentive to induce all eligible engineers within its territory to become members of the Society. The increased membership that will result from such efforts will, it is thought, do far more than recompense the Society financially.

"The increased power and influence conferred on local sections by the plan suggested will, beyond any doubt, result in a very rapid increase in the number of sections, and it is perhaps reasonable to anticipate that before long the delegation from local sections will become so large and representative that it can with propriety be invested with the function of choosing the Board of Direction and possibly also the general officers of the Society.

"The particular form which activities may take in any local section will naturally depend somewhat on local conditions. In those cities where an opportunity for technical papers and discussions is already provided by another local engineering society of general scope, as is the case in Seattle, the section meetings of the American Society of Civil Engineers may be given over to social intercourse and to subjects relating to the welfare of the parent Society, of its members, and of the Profession at large.

"The Seattle Association, numbering 75 to 80 members, holds monthly meetings at luncheon in the College Club, with a usual attendance of about one-third of its membership. It does not read or discuss technical papers. One subject it has considered is the private practice of engineers while employed as professors in the State University, and their nominal engagement as consultants by State and County officials. For further investigation of this and similar matters, a general committee was appointed, consisting of three members from each of the five or six local engineering societies.

"A similar committee, very recently appointed, is undertaking to devise a plan for the federation of all local engineering and technical societies in Seattle. The hopeful thing about this undertaking is that probably the only feasible way to co-relate these existing organizations is one which will preserve and strengthen the identity of each, thus giving the engineer member the advantages of a large general society locally and at the same time reserving to him the distinction of membership in a more exclusive society of National repute.

"Certain of our members suggested the plan of merging all these other local societies into the American Society of Civil Engineers, but apparently the time for that has long since passed. This was pointed out in President McDonald's Annual Address, wherein, at the close of his description of the German National Engineering Society, he made the following comment:

"'I have dwelt at considerable length on the abstract of this article because it shows what might have been accomplished in this country had the management of our Society shown the same interest in the Profession at large as it has in the welfare of its members only.'

"The Seattle Association has given some study to the Address just mentioned, and while our members are in favor of maintaining the present high standard of membership in the American Society, they are also heartily in accord with President McDonald's conception of enlarging the membership of the Society.

"Although the Society must presumably, for the most part, remain aloof from joint action with outside engineers, there appears to be no reason why Local Associations of the Society should not be federated with other local engineering bodies, to the end that the Profession may have a fuller recognition in each community. Such federation will tend to render any one unfavorably conspicuous who continues to practice engineering without associating himself with such a federation of his Profession.

"If there is any well-defined desire for a lower grade of membership than we now have in this Society, that want might more appropriately be supplied by the local federation than by altering our own requirements. The local federation can have such grades of membership as it chooses, but members of the American Society of Civil Engineers, when participating in meetings of such federation, will be recognized as members of the American Society of Civil Engineers, thus reserving to themselves and to their Society whatever of prestige that designation may imply.

"On behalf of the Seattle Association of Members and its President, whom I have the honor to represent, I am instructed to urge your favorable action upon the following recommendations:

"1.—That a committee be appointed to draw up, tentatively, such articles as may be required to provide for the formation of local sections of the American Society of Civil Engineers, and for representatives of those sections to attend and have voice at the Annual Meetings and Conventions.

"2.—That the articles be so drawn as to encourage the local discussion of matters relating to the advancement of the Profession at large, and the general betterment of the members of the Society, rather than exclusively to meetings devoted to technical papers and their discussion.

"3.—That social acquaintance and personal interest among the members of the Society be strongly encouraged.

"4.—That the committee also consider increasing the membership by the methods suggested by President McDonald in his Annual Address at the last Convention.

"5.—It is suggested that the committee be instructed to draw up the said articles and send copies thereof to the Presidents of the existing Local Associations of Members within three months after this Annual Meeting, with the request that the tentative articles be discussed by the Local Associations, and a summary of the views and arguments as well as recommendations, returned to the committee within the next three months. That, again, within the next three months, the committee revise the preliminary articles and send a copy of the revised articles together with a brief summary of the arguments

for and against, to the corporate membership, and arrange for submitting the adoption of the articles to the next Annual Meeting."

THE CHAIRMAN.—Mr. Endicott, do you want to speak for the Washington Association?

M. T. ENDICOTT, PAST-PRESIDENT, AM. SOC. C. E.—If there is nobody here from Washington, I would be glad to say a word. I am a member of the Washington Association. This is a somewhat different organization from the others. It is chiefly social, and it is composed of most of the members of the Society of that city. The dues are $1. It has one meeting a year, which is in the form of a conversazione and a banquet, and the members discuss Society matters in which they are commonly interested; but ours is not a formal organization, such as the others.

That was mooted several years ago. Then there was growing up what is called the Washington Society of Civil Engineers, which is now out of its swaddling clothes, and is a very useful and vigorous society, and that includes the great majority of the members of the American Society of Civil Engineers living in Washington, so that the matter of making ours a more formal organization has been mooted with us, believing that our local association might interfere somewhat with the growth and usefulness of the Washington Society of Civil Engineers. That condition no longer exists, and I think it possible a more formal organization may some time be effected.

THE CHAIRMAN.—The next gentleman.

WILLARD BEAHAN, M. AM. SOC. C. E.—I am President of the Cleveland Association. We have been organized but 30 days. I have come down under instructions to spy out the land and see what was to be expected. We have 45 members. We understand that our territory is limited to Eastern Ohio. Of course, we must not encroach on the territory of Cincinnati. In that territory, as we interpret it, there are 88 members of the American Society of Civil Engineers. We would expect that undoubtedly 75 of the 88 would enter our association. Our dues are $1 a year.

What we have especially in mind is, what our attitude should be toward the Cleveland Engineering Society. That has been answered partly by Mr. Ockerson, and by Mr. Gregory. We have in Cleveland the Cleveland Engineering Society, a society 35 years old, with 750 members, embracing, of course, not only the civil engineers, but the mechanical, electrical, and chemical engineers—we have no mining engineers—and architects. I think it is the most healthy engineering association in this country. I do not think you will find its equal anywhere. The members of the American Society of Civil Engineers, who are also members of that, felt that we should do nothing to duplicate the work, or to hamper the work of our Cleveland association. We would not like to do that; and one of the questions for

which I was to endeavor to find an answer was, what should be our attitude, as members of the American Society of Civil Engineers, toward the Cleveland Society which, of course, is a general society.

We meet every month, and irregularly, about every week, because we have papers for the electrical engineers, papers for the architects, papers for the mechanical engineers, and for the chemical engineers. Although our monthly meetings are general, we have smokers and banquets, and if there is anything that we do not have in Cleveland, it is something that I have not heard of.

I am not particularly concerned about what we should do locally, because, as President, I am afraid we will attempt to do too much. We are very active out there. With regard to legislation along the line of licensing engineers in Ohio, I said to our Committee, merely as a suggestion, that it adopt the policy of President Wilson, "Watchful waiting", and not go too fast.

The Cleveland Association is young, and I have come here to find out what our attitude should be toward the Cleveland Engineering Society, and what the experience of other members has been toward a local society which seems to be filling the whole field of engineering most successfully, and in a way of which we are very proud.

Secretary Dallis.—In that engineering society, do you include architects and mining, electrical, mechanical, and chemical engineers?

Mr. Beahan.—We include civil, mechanical, electrical, and chemical engineers, and architects; there are no mining engineers there. I do not think we would turn any one out if they would show us they were engineers.

E. F. Vincent, M. Am. Soc. C. E.—The organization of the Colorado Association was effected about 1908. We now have a membership of about 75, the Secretary not having heard from all the old members at the time that I left. That association includes about 87% of the members of the American Society in the State. Although our name is the Colorado Association, we do not aim to limit the membership entirely to the State; and, in fact, sometimes some of our old members move outside of the lines, and still retain their membership. The annual dues are $2. We hold regular monthly meetings preceded by an informal dinner, and afterward usually have a paper or an address on some technical subject, often with lantern slides, and also have a social evening after that, if we get through in time.

These monthly meetings are ten to the year. We do not have any during July and August. We also have weekly luncheons, which are intended, primarily, as get-together meetings, to get acquainted. An engineering architectural association was formed about 30 years ago, but it failed to attract to it the best of the Profession, and after some 10 or 15 years it died a natural death.

Our members, the members of the American Society, after some years, felt that it would be a good idea to have a local organization which would get us acquainted with each other. We found that some members of the American Society, who had lived pretty close together for some years, were not very well acquainted.

I came without any special instructions, but to look the land over and see what could be done. We have up with us the question of a bill for licensing engineers, and four years ago such a bill was introduced in the State Legislature, but I believe was not reported out of Committee. At that time the local society felt that since it was possible for such a bill to come up, it would be well for us to get in line and get something that we might propose, not that we felt like pushing it, particularly, but in order to direct legislation and give a suggestion as to what we thought would be proper if something of that kind were to be put through.

At our last monthly meeting, a week ago last Saturday evening, a vote was taken of the members present, and ten expressed themselves as in favor of pushing legislation looking toward the licensing of engineers. Four opposed such action and five were non-committal.

THE CHAIRMAN.—Mr. Lundgren.

LEONARD LUNDGREN, ASSOC. M. AM. SOC. C. E.—I am the representative of the Portland Association. The association was organized June 18th, 1913, with 43 members. Prior to this date there was an informal organization of members of the American Society in Portland, meeting a few times a year. The Society now has 60 members, and the dues are $2.50 a year. The officers are: George C. Mason, President; W. S. Turner, First Vice-President; John T. Whistler, Second Vice-President; G. B. Hegardt, Treasurer; Charles McGonigle, Secretary, 815 Chamber of Commerce. Mr. E. G. Hopson is the only Past-President.

We hold a meeting once every two months. It begins with a dinner, after which a paper is read, and then we have a business meeting. Sometimes these papers are of such general interest that we throw the meeting open to outsiders, sending out invitations, and advertising in the newspapers. At times we have large audiences at these lectures. At the last meeting we had a lecture on the Columbia Highway, illustrated by photographs in natural colors, which was one of the most interesting talks I have ever listened to on an engineering subject. The idea is not to be exclusive, but to give whatever good things we have to others as well.

I happen to occupy a somewhat peculiar situation. As an engineer of the United States Government, I have traveled through all the Rocky Mountain and Pacific Coast States during the last two years. During this period I have belonged to the Denver Association, the San Francisco Association, and the Portland Association. My headquarters are now in Portland, but I retain my membership in the

San Francisco Association, because I feel that that Association is going to be under heavy expense this coming year, and because I wish to feel at home when I go there.

There is one feature which is somewhat peculiar to Portland. There is an Oregon Society of Engineers, which is firmly established, and has a large membership of men, who do not join the National bodies. On the other hand, there are men in the local associations who, like myself, are traveling around so constantly that they do not care to join the local organizations of engineers. That seems to have been worked out very happily in Portland, as the members of the National engineering societies are accorded all the privileges of belonging to the local society except voting.

The Oregon Society of Engineers has broken away from some of the dry paths engineers are apt to travel, and instead of having talks by engineers, they will not permit a member of their society to give a talk at these weekly meetings. They get men from outside to talk to them. The speaker, being a non-technical man, attempts as well as he can to educate us engineers in problems with which we are not intimately connected, and also attempts to show in what way we can help in these other activities. For instance, I attended one meeting at which a member of the Board of Education spoke on the educational problems of Portland and of Oregon, and then he appealed to us to assist the Board of Education in their various plans. I just cite that as one instance, to illustrate a very unique programme which is being followed by that society.

I have listened to the talks of the other members with a great deal of interest. During my membership, I have followed the creation of these associations. I would deplore it a great deal if the local associations became too active in the workings of the parent Society. If the 13-district amendment is adopted, I feel that we will have all the voice that we are entitled to in the working of the Society. I do not think it is for the benefit of this Society to add more officers for the working out of its problems. I am more or less of an *insurrecto,* and think that our Board of Direction can move a little bit faster, but I do not think that by adding to the number of officers, we can help in any way whatsoever, and I do not think that the local associations should take money from the parent Society. We can meet our own expenses and need no financial assistance. The parent Society is making a good return for the money, as I see it. The Board of Direction is far better equipped to handle the financial end of the Society than we are, and I do not think that we should drain its resources for our own local associations.

J. E. Greiner, M. Am. Soc. C. E.—I am President of the Baltimore Association, which has been organized about 8 months. It is open to all members of the American Society in Maryland. It has one annual

meeting, and as many others as may be called by the Board of Directors, and the dues are $3 per year. Its object, of course, is primarily to take an active interest in the business and work of the parent Society, and secondly, a social interest, a personal interest in members, the local members themselves. The association is too young to have done anything so far, but we felt at the start that it would be a good thing for the guidance of future local organizations if the parent Society should outline a Constitution, and let that Constitution be uniform for all associations.

Secretary Dallis.—About how many members have you?

Mr. Greiner.—We have about 80 members. I think there are about 120 or 130 in Maryland.

The Chairman.—Mr. Greiner, do the members of the Society volunteer to become members of the association? That is, do they write to you to become members?

Mr. Greiner.—Any member of the American Society of Civil Engineers may become a member of the local association by sending in an application, provided he is accepted by the Board of Directors.

Mr. Ockerson.—It seems to me there is an important field of usefulness for the local associations in connection with the conditions which cover the entire country. I am a member of the Special Committee on "Floods and Flood Prevention and Allied Subjects"—whatever that may mean—and I think it numbers fourteen, Mr. Secretary, does it not?

Mr. Charles Warren Hunt.—Yes.

Mr. Ockerson.—We have attempted to hold meetings. At the last meeting three members were present, and we have had great difficulty in getting any results from a great majority of the members.

The remark made by the Los Angeles representative suggested to me that that might be a good subject for the local associations to take up—rainfall and run-off, and matters of that kind—which are important features in the work of this committee. If things of that kind could be taken up by the local associations, by tentative committees, with the general committee of the Society, they could give a local atmosphere to matters of that kind that a committee not thus appointed probably could not. I want to call attention to that, as it might be a very important field of work for the local associations.

Mr. Charles Warren Hunt.—I would like to say to Mr. Ockerson that it seems to me that the proper way to bring about the result which he desires would be by communications addressed to the local associations by the Special Committee, specifically outlining the points raised and the information desired, because I am sure that all the local associations would be glad to respond and give it; but let the initiative come from the Special Committee.

The Chairman.—The gentleman who just came in——

Mr. Charles Warren Hunt.—Is Mr. H. S. Crocker, Vice-President of the Colorado Association. I asked him to come in.

Park A. Dallis, M. Am. Soc. C. E.—First, I would say that the Atlanta Association was organized in 1912. It was a case of necessity, for the American Association for the Advancement of Science was coming to Atlanta, and it was rather a big proposition for the technical men of the city, numbering 152, to handle. Of all the engineering societies, there was at that time a chapter of the Electrical Engineers, the Architects, the Engineering Association of the South, and the American Chemical Society. The members of this Society met and organized the Atlanta Association. About 6 months later, the members of the Mechanical Engineers met and organized the Atlanta Section of the Mechanical Engineers.

We have 24 members of the Society, of all grades, in Atlanta, and 56 in the State. We worked under the old Constitution up to the last meeting in January, and at that meeting we drafted a new Constitution, which has not yet received the approval of the Board of Direction. The first Constitution eliminated all politics in the association, and we found that, without a little politics, we got very little work.

The oldest member was automatically Chairman of the association, and at the last meeting, a President of the association was elected, and I come as President under the new Constitution, and as representative under the old.

We have a very small number of technical men in Georgia; in fact, of the six organizations, there are only 205 members in the State, and we have formed with those an affiliated technical society, in which the identity of any branch organization is not lost, or affected by the by-laws and Constitutions of any of the other societies. It is really an affiliation, at which we have barbecues every year. We are working under a number of difficulties at Atlanta; some of the points have been brought out.

One of the points that strikes me very forcibly is as to what should be required of an expert witness. As one of the societies to which I belong deems it improper for an expert witness to answer a hypothetical question, I appealed twice to the Judge—in a recent case—before answering the hypothetical question, and asked him for the right not to answer any hypothetical question for or against my client, but was forced to answer.

There is a tendency in our section of the country toward the lines of purchasable engineering testimony, which we are finding in all these societies, you might say, in an underhand way. There is a quiet movement with a view of appointing a little later a committee from this affiliated technical society to confer with the city authorities. This committee will be composed of one man from each of the National

organizations, to confer with the city to procure the enactment of laws which will protect the technical men in that section of the country.

We have commenced first with the technical officers in the city. The Engineering Committee of the Chamber of Commerce, which consisted of eight men, six of whom were members of this Society, investigated the city affairs, and found that out of 75 cities of about the size of Atlanta, we alone had a peculiar method of electing our city engineer and chief of construction. As this method was not in accordance with the ideas of the committee, it drew up a report, which was written without any feeling or any fear of opposition, not one of the eight men ever wishing to become chief of construction, or willing to accept it if offered; so that we could draft this report without any fear. It was held up in the Chamber of Commerce, but finally approved. There was at that time a little feeling, which I am very thankful to say has been passed over. But we are needing in our section—and, from the remarks made, I judge that it is the same all over the country—some work on the political end, some committees from this Society and other societies, joint committees, to assist in drawing up rules and regulations, making recommendations to State Legislatures, and also municipal bodies.

What I have said relates to engineering of all classes, and more particularly to civil engineering. We have had a hard time in getting the members to the meetings, have no definite points of membership, as the first organization was made simply for the interests of Atlanta, and we did not feel that we could draw on the State, as we wished, for donations for such meetings.

Now, the Constitution, when approved, will be open. Any member of the American Society, regardless of where he lives, may become a member of the Atlanta Association. The dues are not to exceed $3 for resident members, and $1.50 for non-resident members. This year we have taxed the membership $2. It will require that every member of the Society, who wishes to become a member of the Atlanta Association, shall sign the Constitution and by-laws, deposit it with the Secretary, and pay his dues in advance. One year in arrears will drop him from the rolls of the association.

At a meeting some three Saturdays ago we had ten present, and we discussed this matter. At the last meeting nine members were present, and not knowing what this meeting was for, there was a general conversation as to what would be for the best interests of the association and of our members.

One of the points brought out—and it seemed to be the unanimous opinion of those present—was that a Junior member should have more recognition in the Society than simply being on the rolls. From the fact that other societies have a special pin for Juniors, we would suggest investigation as to whether or not this Society should not have this

special pin for the Juniors. It will put them in a position so that where older members meet them on a train, in our travels, we can make ourselves known. They feel more or less timid in speaking to the older men, and it will bring in the social side of the Society, and encourage the younger men to perfect themselves, and become corporate members of the Society.

Another matter was brought up at that meeting, though possibly it is a little irregular. The members present were heartily in favor of the Society having student branches at different colleges, similar to other organizations, so that the students who, after graduation, are eligible for junior membership, might be familiarized with the workings of the Society. The reading of the *Proceedings* will no doubt do these boys good, and our association would strongly recommend the thorough investigation of the question of student branches.

A MEMBER.—What about that recommendation that you made to the Chamber of Commerce looking to the elimination of politics in the appointment of municipal engineers? It is a thing devoutly to be wished. I would like to hear what you suggest.

SECRETARY DALLIS.—We pointed out the wrong, instead of suggesting the remedy. We showed them what was being done by 75 other cities on the subject, cities having from 125 000 to 250 000 people, which is about the population of Atlanta, and in a number of these my memory is that the chief of construction, the city engineer, is appointed by a commission, that is, employed by a commission. In some cases he is asked to stand the examination of the Government service, and in no case is he elected by popular vote.

A MEMBER.—Do you have an election by popular vote in Atlanta?

SECRETARY DALLIS.—Yes, and that election was repeated just a short time ago by popular vote, and in the drafting of one of the ordinances they put in the words, in the appointment of inspectors, that he "shall be a technical man". The labor unions have combined and have gotten the City Council to strike out those words, and we have to get them in again.

Those little points harass us constantly. They are really nobody's business, unless it is the Association of Civil Engineers, or the affiliated societies, and we are trying to get in behind them and keep out of politics. We are not politicians; we are negative politicians. We are for nobody except the man who is fully qualified to hold the position.

Now, the present Smoke Inspector is a Fellow of the Electrical Engineers, and a member of the Society of Mechanical Engineers, and he holds his position with fear and trembling. Recently they put out Mr. Dan Cary, who had charge of the superintendance of parks, for some political reason. I do not know the details of the matter now, but I do know from hearsay that Mr. Cary had been fulfilling his

duties excellently, and it is quite a regret to know that he is out of his position, not on account of Mr. Cary, but simply from the fact that in these positions we want men fully qualified to carry out the work entrusted to them.

Mr. Ockerson.—As an example, which it may be well for other cities to follow, we secured for Member of the Board of Freeholders who wrote the new Charter for the City of St. Louis, a member of the American Society of Civil Engineers, and when matters relating to engineers were discussed they were referred by that member to the engineers, and we threshed it out and he threshed it out in the Board of Freeholders; and the technical qualifications which appeared in the amended or new charter, are now the same as we have in the Constitution of the American Society of Civil Engineers, which is very satisfactory——

A Member.—That is, for the City of St. Louis?

Mr. Ockerson.—The City of St. Louis; all the engineering boards in the various departments of the city works.

The Chairman.—The Chair will say a word about the Philadelphia Association. This association was organized October 17th, 1912, and includes all members of the Society resident in Delaware, and in Pennsylvania east of the Susquehanna River and its northern branch, including the Cities of Wilkes-Barre and Scranton. Its present membership is 112, and there are about 360 members of the Society eligible for membership. Any member of the Society may become a member of the association by signifying this desire in writing to the Secretary and subscribing to the Constitution and By-Laws which have been approved by the Board of Direction of the Society. The dues are $2 per annum. The association meets in January, April, and October of each year. The association has held six meetings, and at the meeting last evening, there were present Professor Charles D. Marx, the President-Elect of the Society, and 125 members and guests.

The question of licensing engineers came up some time ago, a bill having been introduced in the State Legislature and reported out of Committee, placing the licensing of all engineers in the State of Pennsylvania in the hands of the State Highway Commissioner. As a result of protests, a State Commission was authorized for the purpose of considering the matter and making recommendations to the Governor. Our Philadelphia Association then became very active. The matter was discussed by the association, and it unanimously adopted the resolution "that it is the sense of this meeting that it is unnecessary and undesirable to license engineers." A Committee was appointed which submitted a report to the association, and was instructed to present the matter before the State Commission. I am very glad to report that as a result of the activity of the Philadelphia Association, the State Commission has recommended to the Governor that it is undesirable at the

present time to license engineers. The Philadelphia Association has thus demonstrated the value of such associations. If one State should pass a law, licensing engineers, other States would follow, and it is necessary, in order to prevent such legislation, to have some one on the spot at the time the matter is initiated. The local associations of the Society fulfill such a function in a very satisfactory way.

Another matter that has been under discussion, and that I have been authorized by the Philadelphia Association to present before this Conference, is the licensing of architects. Illinois and New Jersey, and some other States have a law licensing architects. In Chicago, I believe the architects have been endeavoring to make it necessary for any one practicing structural engineering, to take out a license as an architect; the resolution passed last evening by the Philadelphia Association was that this Conference should consider the question of appointing a Committee to take up the question of licensing engineers and architects, and if that is not germane to this Conference, that the Society be asked to consider the question of appointing such a Committee.

Mr. Charles Warren Hunt.—It has been asked; it has been before the Board of Direction.

The Chairman.—We hesitated for a long time in Philadelphia about forming a local association of members of this Society, because most of us were members of the Engineers' Club, and there was a feeling that the formation of such an association would be detrimental to the Engineers' Club. I am happy to be able to say that, instead of being detrimental, it has been highly beneficial. A matter which has developed since this association came into existence is the formation of the Engineers' Society of Philadelphia, an organization composed of the Engineers' Club and the local branch organizations of the National Societies—the members of these societies being affiliated members of the Engineers' Society. This organization covers many of the points that have been here discussed; its by-laws have been very carefully drawn, and provide that where there is an expression of public opinion of any kind, if the Constitution of the parent Society forbids such action, that such expressions or resolutions shall contain a statement to the effect that the affiliated association is debarred from participation by the regulations of the parent Society.

I think that this is a solution of the problems in St. Louis, Cleveland, and a number of other places where the members of this Society affiliate with the members of other societies, to form a strong local engineers' body. We do not lose our identity. Before the Engineers' Society are papers relating to civil, mechanical, electrical, mining, and chemical engineering, and each member has the privilege of attending meetings of other societies. The primary purpose of the Society is to

develop a spirit of sociability and co-operation among all classes of engineers in the community.

I came uninstructed by my association. Our Board of Direction discussed many of the questions which have been presented here, and we were of the opinion that these associations should be made co-ordinate parts of the Society; we feel that an association should bear the same relation to the parent Society that a State bears to the Federal Government; that the time is ripe for the Society to divide the country into sections, each to be known by some appropriate name; that such organizations shall elect their representatives on the Nominating Committee and on the Board of Direction; that they in a way pass on the applications of residents in their territory for admission to the Society; and that they in some way be represented at the Annual Meeting of the Society. In other words, these local associations should be utilized for the purpose of upbuilding the Society and strengthening its influence in the country at large.

Mr. Cappelen.—I would like to add to my previous remarks, that St. Paul has a society called the Engineers' Society of St. Paul, and Minneapolis has the Engineers' Club of Minneapolis, two flourishing societies; both feel that this meeting will be of benefit to them, and particularly to the younger members in the American Society.

Mr. Hall.—I would like to add a word to what I said in reference to the participation of the local associations. I presume it is difficult for some of the engineers near headquarters to realize definitely the feeling that exists in the remote places. I know something about it, as I was five years in New York City before going to Seattle, and those things did not occur to me at all until after I went to Seattle and realized how far away from headquarters I was. We are just as loyal to the Society as we can be, of course. I would not want any other idea to go out; and our only idea is to become more useful to the Society, in fact, and that our members, by closer association locally, may be more influential in the community, and receive more benefit for themselves.

I think that the Engineering Profession is large enough and well enough equipped to make its influence felt in any community, and the only reason that it is not felt is because we are not unified in the community; and I have been pleased to note from the reports given here that that problem is being worked out very much along the lines suggested.

A thing we are undertaking now in Seattle is the federation of the local members of engineering societies so that we can have one large body for a general meeting. When we have that, if a paper is presented by the Chemical Society, notice will be sent to all the engineers of this affiliated society, and those interested may attend. If a structural matter comes up, it will interest the mechanical engineers, and

so on; and I think that if any good is to come from our meeting here, it should be looking toward the solving of some of these general problems, so that our local associations may be definitely tied up to the parent Society.

You will note that what I said about the local associations sharing in the revenues was treated almost in a parenthetical way. We are not caring particularly about that, but the idea is that we have a plan by which every member of the American Society would automatically become a member of the local association. The field for developing a local association is very small, and my idea is that it will more than repay the parent Society by the increased activity and by bringing in engineers who will begin to see that they can get some advantages by being members of the American Society of Civil Engineers.

THE CHAIRMAN.—The Chair has been making notes as you have been talking, and has some seventeen suggestions:

The relation of the associations to the Society;
The support of the associations by the Society;
The powers of the associations;
The matter of each association selecting a member of the Nominating Committee and of the Board of Direction;
The relation to public matters;
The publication of papers;
The power and size of the associations;
Territory covered;
Delegates to the Annual Meeting;
Number of members, minimum and maximum, in each association;
The election of members in the Society;
Expert testimony;
Affiliation with other engineers;
Licensing of engineers and architects;
Co-operation of the associations in the work of the Society;
Student branches;
Junior membership should be given greater recognition.

It seems to me that this meeting cannot, by general discussion, do anything. It is a quarter after twelve. I would suggest that we stay in session until 1 or 1:30, discussing our work generally, but that we appoint a committee of three, perhaps, to draft something, and then adjourn to meet late this afternoon, or preferably at 8 o'clock this evening, at which time the committee will have something tangible to suggest. That is merely put forth as a suggestion. The matter is open for discussion.

MR. LUNDGREN.—Are the delegates empowered to do anything? I know I have not the power. I know there are several other men with

whom I have talked to-day, and it seems that most of us are here to look and listen and to go back and tell what we hear.

The Chairman.—The Chair feels that in matters of this kind we do not want too much red tape. The primary purpose of this meeting is to try to make some recommendation to the Board of Direction. We cannot legislate any new laws into the Constitution. That is a matter for the Society and for the Board of Direction, but it seems to me that after all these men have come here to have this discussion, to go back without accomplishing something tangible would not be right. My idea is this, that we recommend something for consideration by the various associations. Let them discuss it, and let it come back again; and in that way we can perhaps crystallize the views of the various associations, and when these are shaped they can be transmitted to the Board for such action as it deems proper.

Mr. Lundgren.—I think it would be proper for us to get the stenographic notes of what has been said, and each association can talk this matter over and decide if it is desirable to do anything.

The Chairman.—Although the Chair does not wish to force things to be done, quite a good deal of criticism is that we do not act. If we adjourn now we will have to meet again. I do not see that any one obligates this Society to be governed by his action, nor does he obligate the Society by putting on record his thoughts.

Secretary Dallis.—Mr. Chairman, I move: First, that you ascertain the best time for us to meet again. I think some of us have engagements at the theatres this evening. Eight o'clock would suit me, and after you decide on the best time for the next meeting, I will move that you read those seventeen memoranda that you have taken, and ask for general discussion on each point, and that you require at least one man present to discuss each item or give us a few words on them, and then later, during this meeting, or at the close of this meeting, that a committee be appointed, sub-divided as you may see fit, a committee or committees to confer with the Board of Direction in a proper method, as to the points brought out by these different associations.

Speaking for Atlanta, although I was not authorized to do anything special here, I feel sure that 98% of our members would be willing and ready to do what these Presidents decide; and I believe that these associations really need some assistance and co-operation with our parent Society.

A Member.—I was going to suggest that we have with us Mr. Hunter McDonald, President of the American Society of Civil Engineers, and that we would all very much like to hear from him.

Hunter McDonald, President, Am. Soc. C. E.—Mr. Chairman, I do not believe it is necessary for me to say anything on the subject. The members know already my sentiments pretty well with regard to

the formation and conduct of local associations. I think the views expressed here to-day refer principally to the matter of affiliation with local societies. I think it is essential that local engineers should arrange some plan of affiliation in order that they may sustain their local organizations, and, in my judgment, the question of what attitude the members of the American Society should bear toward those local organizations is the one that presents the greatest difficulty.

The next question is the extent to which local organizations can affiliate, can participate in local political affairs. I believe that the delegates present here can make valuable suggestions to the Board of Direction, and the Board of Direction would welcome such suggestions.

Mr. Greiner.—Mr. Chairman, it seems to me that the parent Society should take the initiative, at least, in encouraging these local chapters, rather than the local chapters taking the initiative in that respect; and if the parent Society takes the initiative, there should be something definite as a basis for the local members to work upon. The most definite basis that I can think of is a draft of a Constitution for such associations, outlining the objects and principles on which they are to be organized and worked. If the parent Society, the Board of Direction of the American Society, draws up such a Constitution or draft, and sends it to the different local associations, so that all may have uniformity in their methods and their workings, and encourages the formation of those associations in other cities, I think that would be the first step toward obtaining some definite and good results from local associations.

I cannot see that we are going to get any real benefit from local associations as they are now organized and worked. They have originated, you might say, from local members, who are already members of local clubs. There is always a little jealousy at the start. There is always a feeling that any new organization like that will interfere with the social functions and the technical work of the local club, so that that feeling, to a certain extent, must be offset by some definite object which is outlined right from the headquarters. The members of these local associations of the American Society of Civil Engineers should feel that they are just as much at home in their own town, in the discussion of papers, and taking an interest in the business of the local association, as though they lived right here in New York. Therefore, the first thing that the Board of Direction ought to do, is to initiate something definite for these associations to work upon.

Mention was made about discussing the seventeen points that were raised. That discussion can go on for a year. What we want to do is to act on something definite, not merely to discuss these things; these seventeen points might be referred to the local associations, which are very fond of talking. So let them talk and discuss; but let us get down to something definite and appoint a committee to draw up a recom-

mendation to the American Society, and let the Board of Direction take the initiative; then submit their views to the local associations, and let them discuss it for three months, if need be. I make that as a motion.

(Motion duly seconded.)

THE CHAIRMAN.—I should like to speak on one point that Mr. Greiner has raised, that I think may not have been stated in the exact terms to express the idea which he has in mind. It would seem to me that the worst possible thing that could happen would be for the Board of Direction to draft a Constitution and by-laws for any local association. It would be quite a competent and legal thing for it to formulate—in the quickest and easiest way—regulations under which the local associations must form their own Constitution and by-laws.

MR. A. M. HUNT.—A Constitution and by-laws that would fit them all would not fit San Francisco, also.

THE CHAIRMAN.—As I understood Mr. Greiner's motion, it was that a committee of this conference be appointed to-day to draft our recommendations to the Board.

MR. GREINER.—I agree with that. I want to correct the impression that the recommendation was that the Board should form a Constitution.

THE CHAIRMAN.—I did not so understand your recommendation, because we have to assume that the Board has prepared the general form that the associations adopt; what we need more particularly are regulations covering the relations of these associations with the public, as well as with the Society.

A MEMBER.—I am heartily in favor of that.

MR. OCKERSON.—I am not one of those who are fond of speaking too often, but as a member of the Board of Direction, I want to say that the meeting was called as a meeting of the associations. The Board wanted to get the associations together, to formulate certain common grounds on which they could act. The Board acceded to, and in a measure financed, this meeting, and it expects that this meeting will formulate something on which the Board can take action in regard to local associations. That is right, Mr. Secretary?

MR. CHARLES WARREN HUNT.—I think it is. It seems to me, with reference to the relation between the Society itself and its local associations, that we are in a transition state. A great many of the points that were raised, if they are to be carried out, would require a revision of the Constitution of the Society—the question of student members, for instance. I would like to call attention to the amendment to the Constitution, which is to come before the Annual Meeting to-morrow, and doubtless will go before the Society for approval. I believe it will be carried by a large majority. In my opinion, the increase in the number of districts will rather change the general condition.

It has been stated here two or three times—it has been hinted, at least—that it ought to be the function of local associations to select members of the Nominating Committee, to select their representatives on the Board of Direction. That would be a very perfect method of representation, provided the local associations could be made the same as the districts into which the Society is divided. Now, whether that is possible or not, I think it ought to be looked into very carefully, as I believe that would be the ideal way in which the Society could be represented on the Board of Direction.

We have now only seven districts, and you know how large they are; but even if we multiply the number of them by two it would be impossible to get as many districts as there are local associations now, and their number will increase. Would it be possible to sub-divide the country so that we could have twelve local sub-divisions outside of New York? That question ought to receive very careful consideration.

It occurs to me, speaking from the standpoint of the Board of Direction—incidentally, I am very proud to-day, because I was the originator of the local associations in this Society—it seems to me that it would be exceedingly helpful to the Board of Direction if this meeting could, through a committee, formulate the different things which they deem desirable to be carried out, and let the Board of Direction decide the method by which those things shall be carried out, whether by revision of the Constitution, if necessary, on certain points, or whether by the authority now vested in the Board.

In other words, I think the suggestions should be in general terms and not attempt to cover all these questions, some of which involve Constitutional matters; and I believe that this meeting, in which the various associations are so well represented, ought to present to the Board of Direction general ideas on which it can work; otherwise, it will be almost impossible for the Board to do anything. I notice that there is a general consensus of opinion on many things in various localities. If this body could formulate certain general principles which the Board of Direction can give the local associations power to carry out at the present time, it would be a step in the right direction. It could also suggest to the Board certain other things which might be done by making changes in the Constitution.

Mr. Endicott.—The only way to get at this is by way of a committee, as suggested by the Chairman. Mr. Ockerson's motion is a good one; but I hope that this committee will formulate a consensus of the views of this meeting and send copies of it out to the different associations, so that when the matter comes up for consideration by the Board it will have gotten down to the foundation; when it comes before the Board it will be in better shape for action and will receive better consideration than if a recommendation were launched now after an hour or two.

THE CHAIRMAN.—It seems to the Chair that, after all, we want something constructive, and the Board wants something constructive. The representatives of the local associations are in better position to crystallize the views of this conference than any one else. After we agree to anything it should go back to the associations, and we may get some additional ideas which may be of great value in strengthening the first action of the Board of Direction.

We ought to give the committee some time—the Chair feels that it should have the afternoon—to draft something, and we could meet this evening. As I understand, the delegates were supposed to take one or two days, or whatever time was necessary to prepare recommendations. And though I realize that the Great White Way has its attractions, nevertheless, this conference is too important to the Society to allow that to stand in the way of a meeting this evening. The Chair feels strongly that the committee ought to have the afternoon for its work, and that we should meet at 8 o'clock.

MR. HOUGH.—A committee of five, including the Chair, would probably be a little better than a committee of three, would cover the territory a little better. I move as an amendment to the motion that a committee of five be appointed by the Chair, and that they have the afternoon—four besides the Chair.

THE CHAIRMAN.—Is that acceptable?

MR. LEEDS.—I ask the question whether the situation may not arise that that committee of five may desire more time than this afternoon, so it might be better to have the meeting on Friday morning. Then there would be ample time——

THE CHAIRMAN.—It seems to the Chair that all the committee can do is to suggest something, that the real discussion must come from each of the fourteen delegates; I do believe that a committee—and three is large enough—could draft the findings of this conference. The recommendations may be all wrong, but it can draft something definite for debate, and after this afternoon we can determine whether the committee has additional work to do.

A MEMBER.—Isn't everybody going to stay over until Friday?

THE CHAIRMAN.—There is an intervening time—unless the delegates feel that they are occupied this evening, I suggest that we meet then.

MR. HOUGH.—We ought to get to work and push the thing, because I do not believe any of us have any too much time to spare. For my part, I must return as soon as possible. I think we ought to meet this evening and do as much as possible to-day, and no doubt all of us will be here on Friday, and we can wind up our work then.

MR. LUNDGREN.—Mr. Hunt being so thoroughly familiar with all this, I venture to offer a suggestion that he be a member of the committee.

Secretary Dallis.—Mr. Charles Warren Hunt?

The Chairman.—Quite so.

Mr. Charles Warren Hunt.—I am not a delegate, but I will be very glad to give the committee any assistance in my power.

Mr. Ockerson.—Do I understand you accepted five or not?

The Chairman.—The motion is that a committee of five be appointed, four with the Chair, to draft resolutions which will be considered at a session at 8 o'clock this evening. Those in favor of the motion signify by saying "aye"; contrary minded, "no".

It is agreed to.

Now, what is your pleasure? Do we want to continue in session a little longer discussing these matters? Mr. Hunt may have some suggestions to offer.

Mr. Charles Warren Hunt.—I have only one suggestion to make, that, for the information of the members of the Society, a full record of this meeting, after revision by each of the speakers, be printed in *Proceedings*, together with whatever resolutions are adopted by this meeting afterward. This discussion has been very interesting, and would be interesting to every member of the Society, and I think that ought to be done. I cannot make a motion, not being a member.

Mr. Hall.—I move that that be done.

(Motion seconded.)

The Chairman.—I will state the motion. Mr. Hall moves that it is the sense of this meeting that the report in full of this conference be published in the *Proceedings* after editing by the Secretary and after revision by the speakers.

Mr. A. M. Hunt.—Would it be competent to offer an amendment that instead of being published in the *Proceedings,* that it be published separately in such a way that it can be handled separately? Say, for instance, at the meeting of the local associations, I think it would be very much more convenient and very much more available if it were printed separately.

Mr. Charles Warren Hunt.—I accept the amendment, to the motion that I did not make, if you will make it both ways, to be published in *Proceedings,* and also in separate form.

(Motion duly seconded.)

The Chairman.—Those in favor of the motion signify by saying "aye"; contrary minded, "no".

It is agreed to.

Now, gentlemen, shall we have some further discussion or shall we adjourn?

A Member.—Why not leave the work to the committee?

A Member.—I move that we adjourn until 8 o'clock this evening.

(Motion seconded and carried.)

Adjourned.

New York, January 19th, 1915.—Evening Session.—The Conference of Presidents of Local Associations met at 8 P. M., Richard L. Humphrey in the chair, Park A. Dallis, Secretary, and present, also, Messrs. A. M. Hunt, Cappelen, Hough, Jonah, Leeds, Hall, and Vincent, pursuant to adjournment.

THE CHAIRMAN.—Gentlemen, your committee has been at work all the afternoon drafting these recommendations. I am going to ask Mr. Hunt to explain some corrections in them.

MR. A. M. HUNT.—The copies that have been made of the recommendations of the Committee this afternoon require two corrections which have not yet been inserted in the copies that you have. Under Section II, to Paragraph *g*-1 there should be added the sentence, "The President and the Secretary-Treasurer shall be elected by votes of all members of the district."

A MEMBER.—Excuse me, that means all grades of membership as well as corporate members; in other words, does that include junior members?

THE CHAIRMAN.—We have made no distinction there. We do say, Mr. Hunt, in another section, that "all members of any grade without payment of additional dues shall be members", but we do not decide what their voting power shall be. I presume their voting power would be that of corporate members. We might leave that for the Board to decide, but I think, to make it clear, that we should say by the votes of the corporate members of the district.

MR. A. M. HUNT.—Under Section IV, "The Relation of the District Organizations and Local Sections to the Public", add to Paragraph *a* the words "with such organization".

THE CHAIRMAN.—Such "organizations", plural.

MR. A. M. HUNT.—Plural it should be.

SECRETARY DALLIS.—That is, the third section?

MR. A. M. HUNT.—No, under Paragraph *a*, "with such organizations". With those corrections I think the report as you have it in your hands is the recommendation of the Committee.

THE CHAIRMAN.—Now, of the fourteen men who are here representing local associations, there are present nine; is that right? There are fourteen altogether. Mr. Greiner and Mr. Lundgren will not be here, and Mr. Gregory will be here at 8:30, I understand. Mr. Beahan is not here. I do not know whether he will be here or not.

A MEMBER.—There is another man, Mr. Hawley, of Fort Worth, the Texas Association.

SECRETARY DALLIS.—There were fourteen of us this morning.

THE CHAIRMAN.—That was because we counted in Col. Ockerson.

A MEMBER.—I counted sixteen this morning.

THE CHAIRMAN.—That is because there were two from St. Louis and the gentlemen from Colorado and Tennessee. We had Messrs. Hunt,

Hawley, Gregory, Cappelen, Hough, Jonah, Leeds, Hall, Beahan, Vincent, Lundgren, Greiner, and Dallis, and the Chairman; but there are three who are not delegates.

A MEMBER.—That reduces the number to fourteen.

THE CHAIRMAN.—Yes.

SECRETARY DALLIS.—Does not Col. Ockerson represent St. Louis?

MR. JONAH.—No, I represent St. Louis, but Col. Ockerson explained the workings of our organization, as he is an older member than I am.

THE CHAIRMAN.—There are fourteen of us altogether. Now, gentlemen, what is your pleasure with these recommendations?

MR. A. M. HUNT.—It would be a very good plan if you would outline briefly the general plans adopted by the Committee this afternoon on which these recommendations were based, and the purpose of them.

THE CHAIRMAN.—The Committee felt that our recommendations to the Board of Direction should not go into too much detail, that it would be better to touch only the salient features and leave out all details; the idea being that whatever we agree to will be referred to the local associations for consideration, and that the recommendations with the discussion by the local associations should go to the Board of Direction. The thought was that, inasmuch as the Society will be redistricted, as in all probability the amendment offered to-morrow will go through, it would be desirable to create for each geographical district thus created, a district organization under which would come the present local associations; the district organization reporting to the Society and the local sections corresponding to our present associations, reporting to the district organization. The next logical point was the relation between the parent Society and each district, and we covered this, as you will note, by providing that the representatives should be elected by the direct vote of the members in the district which is intended to give direct representation; and this will probably mean that the members of the Nominating Committee and of the Board of Direction will be elected by the votes of the local members, the officers at large being nominated by the Nominating Committee and elected by the general vote of the Society.

Communications affecting the welfare of each district must go through the officers of the district; the Society shall support the district financially by meeting its expenses; it is also recommended that papers presented before the local sections or district organizations, shall be forwarded to the Society and, subject to the regulations of the Society, shall be published in the *Proceedings*; all other communications which have the approval of the district organization shall be printed by the Society in its *Proceedings*. It is recommended that the Board of Direction prepare a Constitution which will cover the regulations governing all districts; that the district by-laws shall be submitted to the

Board of Direction for its approval. The by-laws of the local sections shall be prepared by the section, and approved by the district organization. The Committee believes that there should be a limit to the number of members requisite to form a local section. The Committee is also of the opinion that there should be no allotment of funds to meet the expenses of the local section.

The recommendations further discuss the relations of both the district organizations and the local sections to the public, and permit the affiliation with branch organizations of other national societies and other local engineering societies and clubs; they prohibit, however, the local section from joining in joint resolutions with other organizations; as individual members they can participate in the resolutions; the local section may itself act in matters of local interest. If, however, the matter affects the Engineering Profession, the matter must be transmitted to the Board of Direction before such action is taken; it is the intention that the district organizations and local sections shall act as public agents of the Society in promoting the influence of the Society; and in reporting to the Society any local matter which may affect the Society or the Engineering Profession, in order that proper action may be taken.

This is the general scope of these recommendations on which the Board of Direction is expected to take such action as it may deem proper. The programme is that each local association discuss the recommendations and that before we adjourn, we provide some way in which these discussions and recommendations may be correlated and transmitted to the Board of Direction.

The matter is now before the Conference, and the Chair would like to know whether it is desired that the recommendations be read section by section.

Mr. Hall.—I suggest that we read this, section by section, and give it all the consideration that the members present desire.

The Chairman.—I suggest that our Secretary read the draft, unless Mr. Hunt would like to read it.

Mr. A. M. Hunt (Reading).—

"The conference recommends the adoption of the proposed Amendment to the Constitution, re-districting the Society. Assuming that this Amendment is adopted it recommends the adoption of the following:

"I. Creation of District Organizations and Local Sections.

"*a.*—That there be created a District Organization for each geographical district created by the Amendment above referred to.

"*b.*—All members, of any grade, without the payment of additional dues, shall be members of their District Organization.

"*c.*—It is recommended that the present Local Associations be designated as Local Sections, and

"*d.*—That short descriptive designations be adopted for the District Organizations, and for the Local Sections."

Mr. Leeds.—I would suggest that as each one of these sub-heads is read, unless there are any objections thereto, that they stand approved. Then we will clean up each one as we go along.

Secretary Dallis.—I second that.

The Chairman.—It is moved and seconded that each section be adopted or amended as it is read, that is, be considered as adopted. Those in favor of the motion say "aye"; contrary minded, "no".

It is agreed to.

Mr. A. M. Hunt.—I think that, having read these first four paragraphs without that in view, it would be competent to ask whether there are any amendments or objections to them.

Mr. Cappelen.—I move that Section I, including Paragraphs *a, b, c,* and *d,* be adopted.

(Motion duly seconded.)

The Chairman.—Those in favor of the motion signify by saying "aye"; contrary minded, "no".

The recommendations are adopted.

Mr. A. M. Hunt (Reading).—

"II. The Relation of the Parent Society to the District Organization.

"*a.*—District representation in Society affairs shall be by the vote of the members in the district."

The Chairman.—We might explain that that applies to members of the Nominating Committee and also of the Board of Direction; that is, that the member of the Board of Direction representing the district shall be elected by the members in that district.

The Secretary.—I suggest that the word "corporate" members be placed there.

The Chairman.—Yes; "shall be by the vote of the corporate members". I was going to say, when Mr. Leeds made his motion about each paragraph, that we read right straight through each section, taking from *a* to *i,* instead of taking each separately, because it gives a better idea if you take it all through than by reading each sub-division separately. Unless there is objection we will adopt these recommendations by groups.

Mr. Cappelen.—Yes, by groups.

Mr. A. M. Hunt (Reading).—

"*b.*—Communications from the Society to its members in all matters relating to the District shall be through the officers of the District.

"*c.*—The Society shall not rebate any portion of the dues of the District Organization, but its Board of Direction should make an allotment to cover the business expenses of this organization.

"*d.*—1. The Constitutions of the District Organizations shall be identical; shall deal with basic principles affecting the general welfare of the Society, and shall be drawn by the Board of Direction of the Society.

"2.—By-laws covering local conditions shall be drawn by District Organizations, and shall be approved by the Board of Direction of the Society.

"*e.*—Technical papers presented before the District Organizations or Local Sections shall be published by the Society, subject to its regulations.

"*f.*—All communications that have received the sanction of the District Organization shall be published by the Society for the information of its whole membership."

I think it might be proper to remark, in connection with that paragraph, that the term "communications" there used is distinct from the matter of papers. It refers to resolutions and to declarations of public policy.

Mr. Hall.—Reverting back to Paragraph *b*, "Communications from the Society to its members in all matters relating to the District shall be through the officers of the District", that is, in distinction to general Society matters which go out through the Secretary to the individual members?

Mr. A. M. Hunt.—Yes, sir.

"*g.*—1. The District Organization shall consist of a President, as many Vice-Presidents as there are Local Sections in the District, and a Secretary-Treasurer.

"2.—The office of the Secretary-Treasurer shall be the office of the District Organization.

"*h.*—The Annual Convention of the Society shall be held in the districts in turn; the locality for holding the Convention shall be determined by the Board of Direction upon the recommendation of the District in which the Convention is to be held, except as may be determined under extraordinary conditions by the Board of Direction.

"*i.*—When any question as to the power of any District Organization or Local Section to act on any matter shall arise, the Board of Direction shall, upon the request of the District Organization, decide the matter."

Mr. Hough.—It is only a suggestion that I have in connection with this: In Paragraph *g*—1, "the district organization shall consist of a President, as many Vice-Presidents as there are Local Sections in the District, and a Secretary-Treasurer", why cannot that be fixed up something like this, and save a lot of work: Each section has a President, and these automatically form a board, and this board elects a President and Secretary-Treasurer from its number.

Mr. A. M. Hunt.—The purpose of having the President and Secretary-Treasurer elected by the general membership was this: It may be that there will be members of the Society in the district who will not care to become members of any local association. They, however,

should have representation, and it was written in that way with the idea that there would be a Vice-President, practically, from each local section, but if there was any outside of the local sections they would have a voice in the election of the President and Secretary-Treasurer of the district.

Mr. Hall.—It would probably mean that the President of the local section would become, *ex officio,* Vice-President of the district.

The Chairman.—It does not necessarily follow. That is a matter to be decided by the local section.

Mr. Leeds.—Would it not be wiser not to leave it to the local section; let there be a general rule for the entire Society?

The Chairman.—It seemed to the Committee that that is a detail that might require subsequent action. It has to be promulgated by the Board. It is intended that each section shall have a representative, elected by all the members in the district, and that there shall be two other representatives, thus each member of the district organization, whether or not he is a member of a section, shall have an opportunity for voting for an officer of the district organization.

Mr. Leeds.—It should be with the idea of simplifying the whole matter at this time.

Mr. Cappelen.—It seems to me that Section II is fine, and I move its adoption as a whole.

The Chairman.—Is the motion seconded?

(Motion duly seconded.)

The Chairman.—Is there any debate? Those in favor signify by saying "aye"; contrary minded, "no".

The section is adopted.

Mr. A. M. Hunt (Reading).—

"Section III. Relation of the District Organization to the Local Sections.

"*a.*—A Local Section shall consist of at least 25 members.

"*b.*—Communications from any Local Section to the Society shall be through its District Organization."

Mr. Hough.—Now, Mr. Chairman, I don't think that we ought to fix the number as high as you have fixed it in this article, as to what should constitute a Local Section. That is true, particularly in the West. Spokane could not have made a start three years ago had the requirement been 25 members. Tacoma cannot make a start now. There is no reason why Tacoma, with 15 or 20 members, as I believe they have—I believe I counted 15 to-night in the Year Book—should not be able to start a section and build it up in the same way that we built ourselves up. We are building it larger all the time. We now have 30 members. We intend to increase that number decidedly within the next year or so. We have some members who will come in in a short time, to increase our membership, and Tacoma might

do exactly the same thing, but under this they would not be allowed to start at all; and it seems as if that ought to be left so that the local section could fix a number, that is, say whether they ought to have an organization there, or leave it to the parent Society to say.

THE CHAIRMAN.—What is your suggestion?

MR. HOUGH.—I should put it 15.

THE CHAIRMAN.—Would it meet your views if we said that a Local Section shall consist of at least 25 members, unless the District Organization shall approve of a lesser number, but in no case less than 15?

MR. HOUGH.—Yes; that would be all right. I should leave it optional, so that we can build up, and not try to fix a number which many of the Western towns would never have.

MR. A. M. HUNT.—The only objection to leaving practically no lower limit is that it would tend to increase the number of representatives on the Board of Direction of the District, and that even two or three, if the limit was thrown off entirely, might club together and say, "We will demand a local organization and have as full representation on the governing body of our District as an association with two or three hundred members."

MR. HALL.—Since Mr. Hough has brought the matter to our attention, I am inclined to agree with him. I know the conditions in Tacoma, and considering that our local associations meet often at luncheon and at spare times, it would be very inconvenient to compel them to come over and meet with us in Seattle; and I would certainly be quite willing that they have an association of their own.

THE CHAIRMAN.—I was going to say that we might put after that, unless otherwise provided by the district organization, and that the district organization say what the minimum should be; that is, no local section could be organized with less than 25 men without the approval of the district organization; that is all right; fix it in some way so that these sections can be organized.

MR. A. M. HUNT.—I think that definite provision could be made so that an association could be organized anywhere, but I do not feel that an organization with 15 members should be as powerful in the control of affairs in the district as one of larger membership; if we are going to reduce even to the limit of 25, it seems to me small.

THE CHAIRMAN.—That is true, Mr. Hunt, but you know that in the United States Senate each State has two representatives——

MR. A. M. HUNT.—They have the House of Representatives as well as the Senate, in that case.

MR. HOUGH.—You will not have an association in Montana for a good many years, and you will never have one in Idaho in the lifetime of any of us here; and you will have only two or three in the State of Washington.

Mr. A. M. Hunt.—I am not taking any stand as arguing against the formation of the section, but simply how it will be viewed by some of the local sections of larger membership, where a practically equal voice in the affairs of a district is given to a small as well as a large group. I think it is a matter worthy of consideration.

The Chairman.—I think so, too.

Mr. Vincent.—As I understand, this plan is for the purpose of creating local organizations, or at least, inducing them to organize; as our deliberations here to-night will only result in suggesting to the Board of Direction certain recommendations, we will probably have a chance to thresh that out with them, and they will probably consider it very seriously before any definite action is taken; but, being from the West, where some of the rest of our friends are, and knowing as well as I do how few places there are for some of these organizations, I feel as though 25 is a good many, and at the same time I agree with Mr. Hunt that we do not want to give the local organizations too much authority, too much representation.

The Chairman.—It seems to me, gentlemen, that if we put in the hands of the district organization the right to grant permission for the local sections to organize, they are not going to grant permission if it will lower the voting power of the other local sections, and it seems to me that it is a matter which might be left to the district organization, fixing a minimum of 15.

Mr. A. M. Hunt.—I suggest that we cut out the numbers and say, "A Local Section may be organized wherever authorized by the District Organization."

The Chairman.—There should be at least 25 members. If there are localities where this is not possible, give the district organizations the right to reduce the number to 15; and surely we ought not to attempt to organize any local section with less than 15 members.

Mr. Cappelen.—May I ask: suppose there are 10 members in Montana, 7 members in South Dakota, and 8 members in North Dakota, who ask the district of the Northwest Society to become members, there is absolutely nothing to prevent these gentlemen from coming in. They are not forming a local Society at all; they are simply becoming members of the district.

The Chairman.—Quite so; they become members automatically. The question of the number of members, as far as the locality is concerned, ought to be left to the district, except that we ought to fix a minimum limitation.

Mr. Cappelen.—Yes, we ought to fix a limitation. There is nothing to limit Montana, South Dakota, North Dakota, or any of these States from coming into one district, as long as they do not form a separate organization, and that I understand is not the idea. The idea is that

these members of the American Society want to come into one district and help the whole district along.

THE CHAIRMAN.—They come into a district automatically; they have no volition in the matter.

MR. CAPPELEN.—To me it looks as if they ought to have a representation of 15 at least before they become a local organization.

THE CHAIRMAN.—Answering Mr. Hough: if you cannot get together 15 members you should not have a local section.

MR. HOUGH.—You must have a city of 100 000 inhabitants to muster 15.

MR. A. M. HUNT.—I move that the number be changed from 25 to 15, adding the words "unless by special recommendation of the Board of Direction of the District".

THE CHAIRMAN.—The Chair would suggest that the idea of 25 should be fixed. The local section should consist of at least 25 members, except under special conditions, when the district organization may reduce this number to 15.

MR. LEEDS.—I move that as a substitute.

MR. HOUGH.—I second that motion.

MR. HALL.—I am in favor of that motion, but I want to call attention to one thing. We are really not giving any power to local associations, as such, except the power to organize its membership that elects the district officers.

MR. A. M. HUNT.—They do not elect the Vice-President.

MR. HALL.—They elect him.

THE CHAIRMAN.—If you have a local section, for example, in Seattle, of 100 or 200 members, its influence in the district will be much greater than a smaller section of only 25 members, or of the individual who is not a member of a section.

MR. HALL.—That will not do any harm. It is a matter of detail which will be worked out later; for instance, if a local organization has power to vote in the districts as an organization, of course, it becomes of some importance, but there is no provision for that, so far; their vote is as individuals.

MR. A. M. HUNT.—No; but the Board of Direction of the District will meet the outside matters as the Board of Direction of the Society; and its Vice-Presidents will probably be the Presidents of the local sections.

THE CHAIRMAN.—In other words, 10 men would have as much power as any section of 100, and that is not a fair proposition. It seems to me that we ought to indicate to the Board that it ought to be 25, and never less than 15.

SECRETARY DALLIS.—I have had a little experience along this line in connection with the Civils and Mechanicals. We organized the Civils with a membership of about 15, and the Mechanicals with a

membership of about 12; and I think 12 is rather small. We have had a hard struggle, but we are coming out. I think 18 is small enough for a tentative organization, and in these recommendations we should state that a section should not be recognized as a full-fledged local section unless there are 25 members, but that each district be permitted to grant a tentative organization in special cases where it meets with its approval.

Mr. Leeds.—We have to hit a mean in this matter. If we require that all the sections shall be 25, we may be putting too much of a handicap on the small organizations. On the other hand, if we permit any small number of members to organize a section anywhere, we give too much power to the very small section. On the other hand, if we are going to allow them to have a vote, then the organization becomes too complicated, so that it seems to me that the motion as given is the best, because it sets 25 as a fair number.

Now, if a certain section wants to form with a less number, they then have the burden of proof put upon them that they are worthy of receiving the same ability to vote as the section of 25 or more, and, if the district organization changes, the one that is going to be hurt by it, if at all, is the one that grants the power. So it seems to me that the motion as drawn by the Chairman is the best.

Mr. Vincent.—It seems to me that it might be well if this matter could be fixed so that local organizations could be formed with as few as 15 members, with the consent of the district organization, and not allow them a vote until they could have 25 members. Possibly that does not look practical to some of you, but it was an idea that occurred to me since the number 25 seems to be the minimum which most of those present think should be allowed a voting representation.

The Chairman.—It seems to the Chair that it is rather difficult to say when a section shall be considered full-fledged; if the district organization shall consider that there is not sufficient reason, or that they do not care to authorize a local section of less than 25 members, then the members interested should wait until they can secure the requisite number to form a section. It should be borne in mind that, even with 25 members, it takes considerable effort to keep the section active, and that to create sections which soon become inactive or pass out of existence is not desirable. Sections ought not to be formed that do not have enough members to keep them properly active or in existence.

Mr. Cappelen.—I agree, absolutely. We are here to-day asking for certain concessions from the Board of Direction of the American Society of Civil Engineers, and it seems to me that if the Board will grant us what is provided here, we can go back home and report certainly that we had a very successful meeting.

You are going too far when you ask for little bits of local organizations that would not add a thing, and you are going to give them voting power in the districts, and through the districts in the Society. That is far beyond anything that we ever expected, and I believe that if you will adopt this paragraph just as it is, I think it will be for the benefit of everybody concerned. I make the motion that the section be adopted as it stands.

THE CHAIRMAN.—The Chair would state that there is already a motion before the conference.

MR. CAPPELEN.—I make this as an amendment to the motion.

MR. VINCENT.—I second that.

THE CHAIRMAN.—We have a motion with an amendment. The amendment practically strikes out the motion, leaving the section as it was originally. Is there any debate on that?

MR. LEEDS.—It seems to me that it should stand as the motion was previously made.

THE CHAIRMAN.—Yes; in other words, Mr. Cappelen's amendment is to destroy the motion.

MR. CAPPELEN.—Yes.

THE CHAIRMAN.—The amendment originally offered, Paragraph *"a"*, was amended to read: "A Local Section shall consist of at least 25 members; except that the District Organization shall have the power to authorize a Local Section having a lesser number, but in no case less than 15 members." That was the substance of it. Mr. Cappelen moves to strike out that amendment, which I think is unnecessary, because we can vote on the amendment and defeat it, and that would leave it as it was originally, that is, if that is the sense of the meeting.

The Chair feels that Mr. Hough and some of the Western men have something on their side; in some of the outlying districts it is very difficult to start a section, and it seems to me that if the district organization is willing to divide up the voting power by the creation of a section of a lesser number of members they should have the power to do so, if it is not obligatory on those affected. They are the ones who ought to have the right to decide whether such action is desirable.

It seems to me we might offer the suggestion to the Board, to do as it sees fit. The question is on the amendment as moved.

A MEMBER.—Will you state it?

THE CHAIRMAN.—The amendment is to give the District power to create a local section of not less than 15 members, at its discretion. Mr. Cappelen withdrew the motion——

MR. CAPPELEN.—The motion is right back on the question.

THE CHAIRMAN.—We are voting on the amendment to Paragraph *"a"*. The Chair will call for the ayes. Those in favor of the amend-

ment signify by saying "aye"; contrary minded, "no". The Chair thinks the ayes have it.

The paragraph as amended is agreed to.

Mr. Hunt, will you read the remaining paragraphs?

MR. A. M. HUNT (Reading).—

"*b.*—Communications from any Local Section to the Society shall be through its District Organization.

"*c.*—No portion of the Society funds allotted for District Organization expenses shall be applied to the expenses of a Local Section.

"*d.*—The by-laws of a Local Section shall be approved by its District Organization."

With reference to those three, no action has been taken.

MR. HALL.—I move the adoption of Section III, with its four paragraphs, as amended.

MR. HOUGH.—I second that motion.

THE CHAIRMAN.—All those in favor signify by saying "aye"; contrary minded, "no". The ayes have it. It is agreed to.

MR. A. M. HUNT (Reading).—

"Section IV. The Relation of the District Organizations and Local Sections to the Public.

"*a.*—A Local Section may affiliate with other local technical organizations, but must not, as a body, take action on any matter with such organizations.

"*b.*—A Local Section, as a body, may take action on matters of local interest, but must not act on matters of a National character affecting the Profession at large without the approval of the Board of Direction."

I think, Mr. Chairman, that that should be amplified to express the exact meaning of the Committee, "without the approval of the Board of Direction of the Society".

THE CHAIRMAN.—Yes; "without the approval of the Board of Direction of the Society".

MR. A. M. HUNT (Reading).—

"*c.*—It shall be the duty of each Local Section to report to the Society through its District Organization any matter affecting the general welfare of the Profession.

"All of the foregoing recommendations can be carried out without an Amendment to the Constitution, except in so far as they relate to the representation of each District in the Society.

"It is recommended that the Board of Direction take the necessary steps to amend the Constitution so as to permit the procedure recommended for District representation.

"Respectfully submitted,
"JOHN E. GREINER
"JOHN L. HALL
"A. M. HUNT
"F. G. JONAH
"RICHARD L. HUMPHREY,
"Chairman."

THE CHAIRMAN.—The Chair will entertain a motion.

MR. HALL.—I move that Section IV be adopted.

THE CHAIRMAN.—Is that seconded?

MR. A. M. HUNT.—I second the motion.

SECRETARY DALLIS.—Mr. Chairman, before you put that, I would like to say that Paragraph *a* in Section IV, ties the association down rather closely with regard to affiliations with other local societies. They should have the right to take action on local matters with affiliated societies the same as they take action locally.

THE CHAIRMAN.—The difference is this, that the members can take action individually in the affiliated organizations, but they cannot take action as a body with the affiliated organizations. There is a distinction there; for example, in Philadelphia, we are forming a society composed of the branch organizations of the National Societies, and at the request of the electrical engineers, we have adopted a paragraph which states that where there is action on a resolution—where the Constitution and by-laws of the National Society do not permit—it shall not be adopted without a statement to the effect that the branch organization as a body of that National Society cannot participate in that resolution.

SECRETARY DALLIS.—We have in Atlanta an affiliated Society taking action on local affairs. The record of that has been sent to the Society, and then noted by the parent Society as to any action we have taken. I do not see myself but what the technical reading of that article ties the hands of the local association in affiliating with the other bodies.

THE CHAIRMAN.—When you speak of the affiliated organization, what is the relation of the individual members to that organization?

SECRETARY DALLIS.—They are members through the association, and are represented on the Executive Board by representatives from the section. The six organizations have six executive committeemen appointed by each organization, and those six men have the power to make by-laws, rules, and regulations, and appoint committees to look into city affairs. We are trying to eliminate politics in engineering, and, for one thing, we took up, not long ago, that is, as an association, we approved a request to get a line of precise levels run to Atlanta and along there. This matter was taken up originally by a commission of one hundred men in the Society. The motion made before that committee was made by a member of this affiliation, also a member of the American Society of Civil Engineers, and also a representative of the Mechanicals, and our Senator was notified that it was the unanimous request of the entire membership. I may be wrong on that, but we want some authority.

THE CHAIRMAN.—The Chair has had a good deal of experience in Philadelphia with such a proposition. We are trying to form an

Engineering Society. The purpose of that, and of this particular section, is to strengthen engineering in communities by bringing all the engineers together, whether they be civil, mechanical, or other engineers, under a common head; there is nothing to prevent individual members from acting in such a society, but it does prohibit the local branch—a local branch of our Society, at least—from joining other branches in a joint action.

Mr. Hall.—To clear up the matter a little, I would say that the purpose of that was to prevent involving the American Society in anything contrary to its policy, by the participation of its local organization. The members may vote as they please, but it is not desirable that a local association of the American Society of Civil Engineers should lose its identity, by reason of its participation in these local affiliated societies. It has to adopt policies in line with the policy of the American Society as a whole.

Mr. A. M. Hunt.—I think that I may throw a little light on the matter by saying that the intention was this—or the thing desired to be avoided, rather, was this—that the membership of a local association of civil engineers, participating as an affiliated body, might be overruled by a majority, none of whom was a member of the American Society of Civil Engineers; in which case, if the action were taken as a body, it would be represented as the action of the local association of the American Society of Civil Engineers. The individual members of the local association may vote on any action to be taken by the affiliated body; and if the sense of the local association is adverse to that, the local association could then meet and express its own views as a body.

Secretary Dallis.—I think the intent is in accord with my ideas of the proposition; yet affiliation in Atlanta does not lose the identity of any of the organizations. The identity of the organization is carried strictly and held foremost. The idea is to give us, if we are strong enough to affiliate with a body, some right; if we are not strong enough to affiliate and hold our own with those bodies, we had better wait until we get strong enough to cope with them.

I believe that the American Society of Civil Engineers would have its interests protected rather well on all the affiliated societies. It so happens that on committees, and on the Executive Committee, although we are only allowed one member, we quite often, by having members in two societies, have more than one representative. At present, of the six men in Atlanta, there are two who are members of the American Society of Civil Engineers; but they are not put on in that way. So I do not believe that by giving us a little right to affiliate, that tying our hands with regard to local matters will really be of service to the Society, but will be a detriment.

Mr. Cappelen.—You should add to this clause simply this, that

"without consideration by the district organization and its recommendation to the parent Society"; exactly the same suggestion for the next paragraph, "*b*", "a local section may take action on matters of local interest, but must not act on matters of a National character affecting the Profession at large without the approval of the Board of Direction", I would insert "without the consent of the district organization and its recommendation to the parent Society".

THE CHAIRMAN.—It seems to me the two things are different. For example, take a city which has the four National Societies, Civil, Mechanical, Electrical, and Mining, and they are joined together as affiliated organizations. If we did not have such a provision, then you would have the condition where three of these affiliated organizations might meet, pass a resolution, and it would go out as one adopted by the associated engineering societies of that place, and state that this organization consists of the affiliated local branches of the Civil, Mechanical, Electrical, and Mining Societies. By implication it would involve the Society.

MR. CAPPELEN.—You could do that without passing it through the District.

THE CHAIRMAN.—Yes, I understand; but this is to allow no joint action with any other society.

MR. CAPPELEN.—Outside of the American Society.

THE CHAIRMAN.—Outside of the American Society of Civil Engineers.

MR. HOUGH.—We already have our joint society in working order, and our committee in that joint society is composed of four members from each of the National Societies appointed by the Presidents of these various societies; for instance, it appears, at the present time, that the affiliated societies are represented by the Mechanical, the Civil and the Electrical. That gives them 12 members on their Executive Committee; but four members are appointed from each society by the President of that representative society, and according to this, we would get tangled up.

THE CHAIRMAN.—I do not see that at all, because, for instance, in the Philadelphia plan, you form the Engineers' Society; our Philadelphia association affiliates with the Engineers' Society, but the Engineers' Society can adopt any resolution of a general character it wants; and every member of the Philadelphia Society participates in that resolution as individuals; the resolution goes out, not as the action of the affiliated organizations, but as an engineering society resolution and our association is not tied up by joint action with any other organization.

MR. VINCENT.—I do not see how you are going to prevent the public from believing that the action of the affiliated societies is the action of the organizations that belong to it; and it seems to me

that the public is likely to think that the Mechanical, the Electrical, the Civil, are each responsible for the passage of such resolutions as may go through; each is a member of it, even if they do not take action as a body.

THE CHAIRMAN.—The by-laws of the association prescribe that if the by-laws of a parent Society prohibit that, the resolution must bear on its face a statement that the branch organization is not responsible for it.

MR. LEEDS.—There is nothing in here, as I read it, that requires that. It seems to me that your requirement in Philadelphia is an excellent one, but as it is here, it seems to me that the public, as Mr. Vincent has just said, might get the same idea, placing the responsibility for a certain line of action on the American Society of Civil Engineers, without considering whether they took it as individuals, or as a body.

THE CHAIRMAN.—I see.

MR. LEEDS.—The only recourse there, is simply for the members of the local section of the American Society of Civil Engineers to meet, and, as you might say, render a minority report. That would have to be done in every case for the defence of the parent Society, unless some such provision as you speak of being in existence in Philadelphia is actually included here, that it shall be required; but even there, from what I have seen of affairs in Los Angeles, the public very often fails to take notice of anything of that sort. So I think the only redress is the rendering of some actual protest by the members of the local section of the American Society of Civil Engineers as a body.

MR. CAPPELEN.—I still believe I am right. Section III, Paragraph *b*, states: "Communications from any Local Section"; how are you going to get around that?

THE CHAIRMAN.—That is true.

MR. CAPPELEN.—Do you want a local organization to come in and make any resolution or recommendation they want to, according to Paragraphs *a* and *b* of Section IV? And Paragraph *c* states absolutely that you cannot do it, and that is the way it ought to be. Paragraph *b*, of Section III states: "Communications from any Local Section to the Society shall be through its District Organization".

THE CHAIRMAN.—Are you applying that to IV-*a*?

MR. CAPPELEN.—Section IV, Paragraph *b*, states: "A Local Section, as a body, may take action on matters of local interest, but must not act on matters of a National character affecting the Profession at large without the approval of the Board of Direction".

THE CHAIRMAN.—Yes, but *a* provides that a local section may not join other local sections of technical organizations as a body, or treat any matters.

Mr. Cappelen.—They cannot pass any resolution without going through the district organization?

The Chairman.—In the first place, a local section by itself can act on any matter of local interest. It can act on a matter of National interest provided it gets the approval of the Board of Direction of the Society, but it cannot act without it.

Mr. Cappelen.—In Paragraph *c* you say it cannot act at all except through its district organization.

Mr. Hough.—That is simply in the matter of communications between the local sections.

Mr. Hall.—Would it clear up the matter if we should add to it: "That is to say, any action taken by such affiliated society shall not be binding upon the local section of the American Society of Civil Engineers"? That is about the sense of it.

The Chairman.—No; Mr. Charles Warren Hunt, our Secretary, gave us his opinion on that. His theory is that we ought not to act upon matters, whether they be local or of a general character, with any other technical organization.

Mr. Hall.—Does it mean, for instance, if all the societies have one opinion on a matter, that they cannot take action separately, suppose they all have one mind on a subject, and they all take one action on it?

The Chairman.—That is permissible. Let us assume that in an engineering society only the four are affiliated; the Civils could not join the other three on certain matters, but the Civils could take up the matter of licensing engineers, and could go on record with the permission of the Board of Direction of the Society. That action might be identical with that of the other three societies.

Mr. Hall.—That is about the same thing as we are doing in Seattle; we have a Joint Committee from the different societies to consider certain matters, but that committee has not power to bind any one of those societies as a Joint Committee. They may all agree on the same thing, but they must report back to the societies which appointed the Joint Committee. Those societies must take action separately. I think that is the purpose of this provision.

The Chairman.—We might add a paragraph to the effect that action by a local section shall be in accordance with the provisions of Paragraph *b*; that is, the local section can act individually, but not as a body.

Mr. Vincent.—If our local organizations affiliate with the local organizations of other engineering societies, we cannot escape the responsibility before the public of what the affiliated society does—not very well—even though we insert a statement that none of the societies individually is responsible for the action taken. The public

does not know which one or which two or which three of the affiliated societies are responsible for the acts taken.

Mr. Gregory.—With relation to conditions in Louisiana, all the members of the local organizations of the American Society, I believe, are members of the Louisiana Engineering Society. That is a very much larger body, having more than 200 members. All the action in the past, taken in regard to legislation, the licensing of engineers, and that sort of thing, has been suggested by and carried through by the society. Now, we, as members of the Louisiana Engineering Society, act merely as members of that society. We are not committing the American Society of Civil Engineers to any particular policy. We are merely acting within our rights, as members of that local Engineering Society.

For instance, six years ago the society started an agitation in regard to licensing engineers, and without doing anything very radical at that time, they had the Legislature pass a bill which made it necessary within a given time for all engineers and surveyors to register as such. A great many men registered at that time who probably had no right to register, that is, no right to their qualifications as engineers or surveyors. The two grades were created at that time.

At the last meeting of the Legislature, which occurred last summer, a bill was passed which was fathered by the Louisiana Engineering Society, in which were added certain other provisions. I should have said that after the time had expired for registration as a civil engineer, if a man wished to practice civil engineering in the State, he must appear before an examining Board.

The personnel of that Board was made up by appointment by the Governor from a list furnished by the Louisiana Engineering Society. That was a part of the bill. Now, the amended bill is very much like the first one, with the exception that each civil engineer must register yearly with the Board of Examiners and pay a license fee of $3 for practicing civil engineering, or if he be a surveyor, a license fee of $1 per year, and he must register before a certain date.

The income from this is to be used for paying the expenses of the Board, and if a man comes to the State of Louisiana and wishes to practice civil engineering, he applies for an examination, and if he passes the examination a certificate is issued allowing him to practice in the State of Louisiana. This applies only to civil engineers. The question has never been raised, so far as I know, as to the broad interpretation of the term "civil engineers", whether it shall mean everything outside of military engineering. It has not been applied to marine engineering, mechanical, or electrical engineering, as ordinarily understood.

Secretary Dallis.—I still feel that it is more or less a technical point. I also feel that it is almost useless for the Atlanta members of the American Society to affiliate with the other societies with this restriction. The representative would simply feel that he had his hands tied; and the members will not be in a position to take action in regard to local affairs.

In order to affiliate, we ought to have some authority on local affairs. If we have not, I would be in favor of considering not affiliating.

The Chairman.—As the Chair sees it, this permits the affiliation with other organizations in matters of sociability, and other matters, not affecting the general Profession of engineering. In matters solely of local interest where some question is involved, whether of politics or otherwise, there is nothing to prevent the local section of this Society from taking part, or in matters of National importance, with the approval of the Board of Direction.

Secretary Dallis.—It does not say so in this section.

The Chairman.—The Chair thought it read that way.

Mr. Jonah.—The reason for affiliation is usually to obtain a successful club. The membership in one branch of the Profession might be so limited that they could not maintain a successful working club by themselves, but associated with two or more branches of the Profession, they can make a very good club. That is the history of the St. Louis Engineers' Club. When it was a club, restricted in its membership to the civil engineers, it was a small struggling club with an indifferent attendance, but when they affiliated with the mining engineers, the electrical engineers, and the mechanical engineers, they made a very strong club, and I think that is the primary reason for affiliation.

The Chairman.—It was stated in Philadelphia by the Philadelphia Branch of the Electrical Engineers, that the St. Louis affiliation had been unsuccessful, so far as the local chapter of Electrical Engineers was concerned, because that chapter had lost its autonomy and identity and had really retrograded.

Mr. Jonah.—They may have lost their identity, and I cannot speak for the Electrical Engineers. They still have their representation in our meetings. For instance, they have the right to present their papers in rotation.

Mr. Hall.—I do not want to weary you with much speaking, but as I see this, I think it ought to be left just as it is, for without such a provision the local association of the American Society would not be in a position to make a protest on any action taken by the affiliated societies. It might happen that in a meeting our membership would be very poorly represented, or not at all, and something might come up to be acted on, and, when our members got together, they would

see that it was an unwise measure. Without a provision such as this, they would not be in a position to take action. So that, with the amount of time we have had to study the wording, I think it is perhaps as good as we can do.

Mr. Cappelen.—I move their adoption as presented.

The Chairman.—That is, Section IV, the two concluding paragraphs.

(Motion seconded.)

The Chairman.—Those in favor of the motion signify by saying "aye"; contrary minded, "no". The ayes appear to have it. The ayes have it.

Now, Gentlemen, the programme is this: I presume each gentleman will be the medium by which the result of the deliberations of this conference is taken to the local organizations. It is thought that these recommendations should go to the Board of Direction, after they have been edited, and with the approval and comments of the various associations. It is open now for the conference to decide what to do. It is suggested that perhaps the editing of this and the transmission might be left to some one or some committee or otherwise that the conference may designate.

Mr. Leeds.—Before we come to that there is another matter that I would like to speak on, though it is a detail and possibly may be, as such, ruled out of order as not pertinent to this general matter, yet at the same time it seems to me so important that we should make some recommendations to the Board of Direction on it, namely, that matter of the Board of Direction asking the local districts or sections, as may be determined best, at least an opinion as to the qualifications of applicants for membership.

The Chairman.—When this matter was discussed in committee, it was felt that with this plan going through—at least, the Secretary of the Society expressed that belief—that the Board would write to the district officers for information, it would become a regular practice for the Board to write to the district organizations regarding new members in the district.

Mr. Leeds.—Would it not be well to put that on record?

Secretary Dallis.—Well, I am in favor of the parent Society taking this matter up with the districts, and our local organizations; but I am not in favor of having that made as a written recommendation to them, I think they should do that.

Mr. A. M. Hunt.—Mr. Chairman, I would suggest that our Chairman be requested to ask the Secretary of the Society to transmit to each of the local sections a corrected copy of these recommendations which we have in hand, with the idea that they shall be discussed with the local associations; that the reports of the local associations be rendered and returned to the Secretary of the Society; and that the

Secretary of the Society, with Mr. Humphrey, our present Chairman, codify these and lay them before the Board of Direction of the Society at the earliest date possible. I would offer that as a motion.

THE CHAIRMAN.—Is that motion seconded?

SECRETARY DALLIS.—I second the motion.

MR. CAPPELEN.—Is there any objection to submitting this draft to the Board of Direction as it is?

THE CHAIRMAN.—There is no objection, but I think there is a feeling in the Committee, shared by the Secretary of the Society, that it might be desirable to have the local associations discuss this, and that valuable points might be added to these suggestions of the conference.

MR. CAPPELEN.—How are you going to bring it before the Society?

THE CHAIRMAN.—I understand, it is the Secretary's idea that after being codified it would go to the Board of Direction with the statement that this had been adopted by the conference on January 19th, had been submitted to the local associations, and the suggestions were submitted to the Board for its information. In other words, we would have to codify the criticism of each local association on the subject.

MR. CAPPELEN.—Do you want that to go to the Board of Direction or to you?

THE CHAIRMAN.—No, we would use the machinery of the Society to send this out and collate the replies, which the Secretary and the Chairman of the conference would codify and transmit to the Board of Direction; isn't that the understanding?

MR. A. M. HUNT.—That is the understanding.

MR. GREGORY.—Would the Chairman give us the exact wording of Section 3-*a*? I believe that was changed.

THE CHAIRMAN.—"A Local Section shall consist of at least 25 members except that the District Organization shall have the power to authorize a reduction in this number, but in no case shall it be less than 15 members." That is, at the discretion of the district organization, it can be reduced to 15 members.

MR. HALL.—There is one point that, perhaps, ought to be considered at this time. The plan is to send this report to the local associations. Ought it not at some time, either now or a little later, go to all the membership of this Society before it is considered to be final? A good many men would be affected by this who are not members of the local associations, as they become members of these district organizations, automatically, and possibly they might not be in favor of it.

THE CHAIRMAN.—The Chair is unable to say whether that would be proper. It seems to me, however, that this conference came into existence under the authority of the Board of Direction. It might be desirable to leave to the Board of Direction the discretionary power of sending this to the membership at large, as it sees fit.

Secretary Dallis.—This morning a motion was carried that the minutes of this meeting appear in the *Proceedings,* and go to every member.

The Chairman.—The Chair understands that all that we do will be printed in the *Proceedings,* and also in separate form, and go to the members of the Society.

Mr. Vincent.—In one of the resolutions was there not a time fixed by which this report should be returned after discussion?

The Chairman.—No; I think Mr. Hall's suggestion fixed three months, but that was merely his individual opinion, and was not acted on by the conference.

Mr. Vincent.—Might it not be well for us now, in this motion, to have an understanding as to the time by which this action shall be considered, and returned?

The Chairman.—The Chair was going to suggest that, but he thinks, perhaps, it might be as well to have that as a separate resolution; not to have too omnibus a resolution, but agree to one thing at a time.

Is there any further discussion of Mr. Hunt's resolution, which, in substance, is to have the Secretary of the Society send this to the local associations with the request that they transmit any criticism to him, and that the codified replies, discussions, and recommendations be transmitted by the Chairman of the section to the Board of Direction.

Mr. Jonah.—I think that the Secretary, when he sends that out, should request the return of the criticisms within a certain length of time.

The Chairman.—My thought was that we should not decide now. Let us discuss the date when the replies should come in. Is there any further discussion on Mr. Hunt's motion? Those in favor of the motion signify by saying "aye"; contrary minded, "no".

It is agreed to.

I judge from the remarks this morning, that most associations have fixed times for meetings. The Philadelphia Association had a meeting last night, and they do not meet regularly again until April. It seems to me that this important matter might be made a special order of business at a special meeting called to consider this alone; and it seems to me not unreasonable that the reply be required to be returned within thirty days.

Mr. A. M. Hunt.—I would suggest a definite date. I would suggest March 15th.

The Chairman.—Will you make that as a motion, Mr. Hunt?

Mr. A. M. Hunt.—I will offer that as a motion, that the Secretary of the Society be instructed, in sending out these communications, to state that any criticisms to receive consideration must be in the hands of the Secretary of the Society not later than March 15th.

THE CHAIRMAN.—May I suggest that that be a little more mandatory, that each association send its approval, disapproval, or comments on these recommendations, not later than March 15th. We ought to hear from every one of the local associations.

MR. CAPPELEN.—I second that motion.

THE CHAIRMAN.—Gentlemen, is there any discussion of the motion?

SECRETARY DALLIS.—I think it would be well, before that is sent in, that the records of this meeting and the *Proceedings* go before the membership; and it is a question as to what *Proceedings* those minutes will appear in, as to whether or not March 15th is a proper date.

THE CHAIRMAN.—What do you mean, what number of the *Proceedings?*

SECRETARY DALLIS.—Yes; do we get this in the *Proceedings* of February?

THE CHAIRMAN.—I fancy it will go in the February *Proceedings*, the Report of the Annual Meeting.

MR. HOUGH.—I do not see why we could not get it right away, if we could have a copy of this, our regular meeting comes on the second Friday of next month. We will not have another meeting for 60 days; and, of course, if we wait until this printed matter comes out, we cannot take action on it. It will be very nearly a month before that comes out, and comes before us.

THE CHAIRMAN.—The Secretary of the Society stated, and I think it was accepted, that each member would have a chance to revise his remarks, after the stenographer had transcribed the notes. The copy has to go to each of the fourteen members to be corrected and returned, and that is going to take time, so that I do not think we ought to tie up the action with the stenographer's report. It seem to me, further, that these recommendations are perfectly clear, and that the representatives here will be at the meetings of the local sections and can explain what took place.

SECRETARY DALLIS.—I would suggest, as an amendment to Mr. Hunt's motion, that the words "to be sent in with comments at a time designated by the Secretary of the Society", be added, and leave that to Mr. Hunt. The purpose is that Mr. Hunt may see that we have all had a chance to look over all the data in regard to this matter, and that we are not pushed for time. Two weeks, or three, would be ample time after we have the *Proceedings*, and the membership has looked them over, to make our comments and send them in, but those minutes may be delayed in getting into the *Proceedings*.

THE CHAIRMAN.—May I ask what relation there is between the transcript of the deliberations of this conference and the ratification?

SECRETARY DALLIS.—So that the membership at large may see the result of these deliberations.

THE CHAIRMAN.—The membership at large is not going to vote on

this. The Chair feels that each member here is just as capable of transmitting to his association the recommendations of this conference as the *Proceedings* could possibly be. We want action, and we do not want to delay this indefinitely. If returned by March 15th that would certainly delay it long enough.

Mr. Cappelen.—The Northwestern Association's regular meeting is to-morrow, but has been postponed. They said they would simply wait until I came back and told them all about it. They said, "Then we will have the meeting and do exactly what you say; you tell us the whole proposition, so we will be ready to have a meeting called and get ready to proceed whatever happens"; and they can act right away on the documents that I have in my pocket.

The Chairman.—The Chair does not want, Mr. Dallis, to stop you from making a motion, but it seems that it would undoubtedly delay it to wait for two months. Are we ready for the question? Those in favor of the motion signify by saying "aye"; contrary minded, "no".

It is passed.

Are there any other matters that the conference wishes to bring before the meeting?

Mr. A. M. Hunt.—Before we adjourn it would be quite proper to move a vote of thanks to the Board of Direction for giving the representatives of the local associations the opportunity to submit these recommendations. It certainly is a recognition of the local associations, and an expression of very keen friendliness on the part of the Board of Direction to them; and I think it is quite worth while to take such action.

Mr. Hall.—I wish to second that motion.

The Chairman.—The Chair thinks it might be phrased a little better.

Mr. A. M. Hunt.—It could be phrased very much better.

The Chairman.—That this conference of representatives of local associations wishes to record its hearty appreciation for the opportunity afforded it by the Board of Direction of the Society to offer suggestions as to the ways in which the activity and strength of the local associations may be promoted.

Mr. A. M. Hunt.—Aye, aye, sir.

The Chairman.—Does that cover your idea?

Mr. A. M. Hunt.—Very good.

The Chairman.—Is that resolution seconded?

Mr. Hall.—I second it.

The Chairman.—Those in favor say "aye"; contrary minded, "no".

It is carried.

Mr. Cappelen.—I move that we offer a vote of thanks to the local committee of the fourteen members who worked to-day and made such a splendid presentation of the facts as they have done.

(Motion seconded.)

THE CHAIRMAN.—It is moved and seconded that the conference records its appreciation of the work of the committee that prepared the preliminary draft of the recommendations.

MR. HOUGH.—I second the motion.

THE CHAIRMAN.—Those in favor signify by saying "aye"; contrary minded, "no".

The resolution is agreed to.

Is there any further business that the members of the conference wish to bring up?

MR. HOUGH.—There is one thing more; that is about re-districting the Society. It seems as if, especially in the western part of the country, the Western members could offer some suggestions to the Board about how that ought to be done. As to the division now, the Western country is peculiarly located, and a certain redivision might make it much better than some others. Some suggestions from the Western members might help them out, and the Board could take them for what they are worth, at least.

THE CHAIRMAN.—The Chair understands that Mr. Hough's motion is to the effect that, should the amendment redistricting the Society be adopted, it is desirable that the Board of Direction shall consult the local sections in fixing the new districts.

MR. HOUGH.—That is the idea.

MR. HALL.—There is some little question of the propriety of a motion of that sort. It seems to me that the Board of Direction, having given us the consideration that they have, would probably not overlook us in so important a matter as that, even if we did not take action on it.

MR. LEEDS.—That is a detail that can as well be left as that matter of membership. The spirit of the Board of Direction is one of co-operation with the local associations.

MR. JONAH.—That expresses the idea I have in mind. We do not wish to do too much suggesting to the Board of Direction.

THE CHAIRMAN.—The Chair did not hear the motion seconded. The Chair would say further that he thinks if we could have these three or four basic ideas adopted by the Society, we would have more than justified our conference; the details matter little if the principle is adopted. It is a vital principle that is involved, which means much for the future growth of the Society. It means a direct representation from the districts themselves, and it gives the outside members of the Society a standing, which they have never had before. I think that principle is too important to be involved by little details which will work out in all probability to our satisfaction.

These recommendations are merely a skeleton, and in years to come, if they are adopted, there will be many amplifications, which will increase the power of each district.

Is there any further business?

Secretary Dallis.—Will there be any other meetings of this conference?

The Chairman.—The Chair does not understand that any other meeting will be necessary. The original belief, I understand, was that we would be in session several days; but I think that was predicated on the assumption that this might be a rather inharmonious gathering, and might not be able to get together; apparently we have been able to discharge our business with much greater rapidity than was anticipated. So the Chair sees no reason for another meeting, unless the members want one; in fact, he does not think it would be fair to continue the meeting indefinitely, unless there is some prospect of new business of importance. We had better adjourn *sine die;* that is, we are out of existence, unless the Board calls us back.

The Chair, in closing, wishes to express to the members his appreciation of their activity. We ought to congratulate ourselves on having accomplished a very valuable piece of work. He hopes that each one will take back to the locality from which he came an enthusiasm that will galvanize that locality into renewed activity.

Those in favor of adjournment say "aye"; contrary minded, "no".

(The conference then adjourned.)

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

May 5th, 1915.—8.30 P. M.—A regular business meeting will be held, and a paper by Charles H. Paul, M. Am. Soc. C. E., and A. B. Mayhew, Assoc. M. Am. Soc. C. E., entitled "Temperature Changes in Mass Concrete", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

May 19th, 1915.—8.30 P. M.—At this meeting, a paper by J. D. Galloway, M. Am. Soc. C. E., entitled "The Design of Hydro-Electric Power Plants", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

June 2d, 1915.—8.30 P. M.—This will be a regular business meeting. A paper by W. L. Du Moulin, Assoc. M. Am. Soc. C. E., entitled "The Pumping Plant of the Morenci Water Company", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Annual Convention of the Society will be held in San Francisco, Cal., September 16th, 17th, and 18th, 1915, being the Thursday, Friday, and Saturday immediately preceding the International Engineering Congress, which is to be held during the week beginning September 20th.

Three of the other National Engineering Societies, under whose auspices the International Engineering Congress is to be held, will also hold meetings in San Francisco at about that time.

Arrangements have been made for a special train, and possibly more than one train, to accommodate the members of all of these Societies who wish to attend their own meeting as well as the Congress.

A general circular containing information as to transportation, hotel rates, etc., has been issued, and later a programme more in detail will be sent out.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed fre-

quently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meetings)

February 27th, 1915.—The Special Meeting was called to order at 1.30 P. M., at the Albany Hotel; President Vincent in the chair; Roger W. Toll, Secretary; and present, also, 21 members.

President Vincent read the minutes of the Conference of Representatives of the Local Associations, held in New York City, on January 19th, 1915. The recommendations adopted by that Conference were generally discussed by those present, and on motion, duly seconded, it was unanimously resolved that the Association does not approve of them.

In the absence of the Chairman of the Committee appointed to prepare a bill providing for the Licensing of Civil Engineers in the State of Colorado, Mr. A. O. Ridgway stated that as there had not been sufficient time to allow the Association to consider the bill, it had been introduced in the Legislature, but not as coming from the Association. After a general discussion of the subject, on motion, duly seconded, it was resolved that the Association is opposed to the passage of any bill to license civil engineers at this time, and that the President and Secretary be instructed to see the proper legislative authorities and urge that the bill be withdrawn or defeated.

On motion, duly seconded, Mr. H. S. Crocker was instructed to advise the Board of Direction of the Society of the action taken by the Association in regard to this bill.

A letter from the Secretary of the Society requesting suggestions in regard to holding one of the Spring meetings of the Society at some place other than New York City, was read.

On motion, duly seconded, the Secretary of the Association was instructed to call the attention of the members to the desirability of becoming members of the International Engineering Congress to be held in San Francisco.

Adjourned.

March 13th, 1915.—The meeting was called to order; President Vincent in the chair; J. Y. Jewett, acting as Secretary; and present, also, 21 members and guests.

The minutes of the Special Meeting of February 27th, 1915, were read and approved.

The Report of the Committee appointed to draft a bill for the Licensing of Civil Engineers was presented by Mr. J. E. Field. On motion, duly seconded, the report was accepted and the Committee discharged.

Communications were read from the Philadelphia, Cleveland, and Portland, Ore., Associations, regarding the action taken by said Associations on the Recommendations of the Conference of Presidents of Local Associations held in New York City on January 19th, 1915.

A paper by Mr. A. J. Allan, on "A Tramp Across an Unexplored Country", describing a reconnaissance survey for the Grand Trunk Railway, south and west of Hudson Bay, was presented by the author and illustrated with lantern slides. A vote of thanks was tendered Mr. Allan for his interesting paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. P. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

The proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on January 6th, 1915.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

The Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.) has been organized with the following officers: Frank M. Kerr, President; J. F. Coleman, and W. B. Gregory, Vice-Presidents; A. M. N. Blamphin, Treasurer; and L. C. Datz, Secretary.

Northwestern Association

The proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.) was considered and approved by the Board of Direction of the Society on November 4th, 1914. F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

At the meeting of the Association on October 5th, 1914, the following officers were elected for the ensuing year: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow, and William Hunter; Directors, John Sterling Deans, John W. Ledoux, Edgar Marburg, and H. S. Smith; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

Portland, Ore., Association

At the meeting of the Association on October 21st, 1914, the following officers were elected for the ensuing year: President, George C. Mason; First Vice-President, W. S. Turner; Second Vice-President, John T. Whistler; Treasurer, G. B. Hegardt; and Secretary, Charles J. McGonigle.

St. Louis Association

The proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on October 7th, 1914.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensu-

ing year: President, R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

(Abstract of Minutes of Meeting)

February 22d, 1915.—The meeting was called to order at 12.15 P. M., at the College Club; Vice-President A. S. Downey in the chair; Carl H. Reeves, Secretary; and present, also, 18 members and guests.

The minutes of the Annual Meeting, January 25th, 1915, were read and approved.

Richard Khuen, M. Am. Soc. C. E., addressed the meeting on engineering and technical matters in the Eastern United States.

Mr. John L. Hall, the Association's representative at the Conference of Presidents of Local Associations, held in New York City on January 19th, 1915, opened the discussion on the report of that Conference, and communications on the subject, from the Secretary of the Society and Mr. S. H. Hedges, were also read.

On motion, duly seconded, the Association unanimously endorsed the general programme and specific recommendations adopted by the Conference of Presidents of Local Associations.

Mr. C. C. More called attention to the present method of election to membership in the Society by the Board of Direction, and suggested that applicants for membership be investigated and reported on by the nearest Local Association, as well as by the endorsers named by the applicant.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 515 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

The proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on March 4th, 1914. Ulysses B. Hough is President.

Texas Association

The proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on December 31st, 1913. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Steel Columns and Struts

March 10th, 1915.—The meeting was held at the Bureau of Standards, Washington, D. C. Present, A. L. Bowman (Chairman), James H. Edwards, Charles F. Loweth, Rudolph P. Miller, and Lewis D. Rights. The following visitors were present: Dr. G. R. Olshausen, Physicist of the Bureau of Standards; Mr. F. Auryansen, Bridge Engineer, Long Island Railroad; Mr. H. T. Welty, Engineer of Structures, New York Central and Hudson River Railroad; Messrs. L. C. Dilks and D. W. Bliem, of the Eastern Steel Company, New York City; and George E. Strehan, Assistant Engineer, Bureau of Buildings, New York City.

The Committee inspected a column, Bethlehem section, slenderness ratio, 120 $\frac{L}{R}$, heavy type, which was being tested in the machine.

The Progress Report, the question of discussions, and the closure of the discussion, were considered informally, and it was decided that no closure should be attempted until more discussion was received.

Various members of the Committee reported concerning the methods of plotting the strains at the different loadings, and the results of tests made to date and the comparison between the ultimates shown by the light and heavy sections were fully discussed.

On motion, it was decided that the Committee present a Progress Report to the next Annual Meeting of the Society, which report should contain full information as to the tests.

On motion, it was decided to call a meeting of the Committee in Washington, D. C., early in June.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 312 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From March 2d to April 3d, 1915)

DONATIONS*

WORKING DATA FOR IRRIGATION ENGINEERS.

By E. A. Moritz, Assoc. M. Am. Soc. C. E. Cloth, 9¼ x 6 in., illus., 13 + 395 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $4.00.

In this work, the author's object has been, it is stated, to produce a handbook or pocket-book for irrigation engineers, which will result in conservation of time and mental energy to the user, and to present material on the subject not readily found elsewhere. The usual steps followed in the development of an irrigation project are briefly discussed in the order of their sequence and this matter is followed by detailed explanations of the tables and diagrams of which the major portion of the book is said to consist. The method of irrigation by gravity has been kept in mind in discussing the various features, it is said, as being the most important, but most of the principles apply also to irrigation by pumping. The author states that a considerable portion of the subject-matter is original, the remainder to a large extent being taken from the publications and records of the United States Reclamation Service, and he hopes that the book will prove of value to irrigation and hydraulic engineers. The Contents are: List of Diagrams; List of Tables; Examination and Reconnaissance; Investigations and Surveys; Design of Irrigation Structures; Hydraulic Diagrams and Tables; Structural Diagrams and Tables; Miscellaneous Tables and Data; Specifications; Index.

LAND DRAINAGE:

A Treatise on the Design and Construction of Open and Closed Drains. By J. L. Parsons. Cloth, 9¼ x 6¼ in., illus., 11 + 165 pp. Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1915. $1.50.

This treatise is offered, it is stated, with the hope that it may prove of assistance to those connected with the design, construction, maintenance, and general administration of land drainage enterprises, namely, the engineer, the contractor, the land owner, and the drainage district official. The author states that the volume has been made as complete as possible on all subjects connected with the successful drainage of agricultural lands by open and closed drains, including surveys, plans and specifications, designs, maintenance, estimates of cost, etc. In the Appendix he has given a copy of the judge's instructions to the jury in the trial of a drainage damage suit in an Iowa Court, which shows the basis on which damages for open drains should be allowed under the Iowa law. There are also tables of historical records of representative tile and open drains. The Chapter headings are: Preliminary Drainage Surveys; The Design of Tile Drains; Tile Drain Outlet Walls and Inlets; Design and Maintenance of Open Drains; Plans, Reports and Records; The Estimate of Costs of Drainage Systems; The Preparation and Enforcement of Drainage Specifications; The Division of Costs of Drainage Systems; The Quality and Inspection of Drain Tile; Appendix; Index.

THE OPERATION OF SEWAGE DISPOSAL PLANTS:

A Manual for the Practical Management of Sewage Disposal Works, with Suggestions as to Improvements in Design and Construction. By Francis E. Daniels. Cloth, 8¼ x 5½ in., illus., 136 pp. New York, Municipal Journal, 1914. $1.50.

The subject-matter contained in this book, it is stated, is based entirely on the author's personal observation of and experience with almost all known types and combinations of units of sewage disposal plants. It was first published serially in the *Municipal Journal*, but at the request of many sewage plant attendants and designing engineers, it has been put into book form with the hope that it will prove of value as a handy and practical manual. The author's main purpose, it is said, is to assist the plant attendant in keeping his plant at its highest efficiency at a minimum expenditure of cost and energy, by pointing out what he

* Unless otherwise specified, books in this list have been donated by the publishers.

should and should not do. He has also endeavored, it is stated, to show that poor designs are troublesome, costly, and inefficient, with the hope that more attention will be given to details which have a direct bearing on operation as well as on the fundamental principles of treatment processes. On account of the lack of reliable every-day data, the book is said to be wanting in detailed results of many kinds of sewage disposal processes and the effects on the various outputs caused by the different types of construction and their different modes of management and operation. The Contents are: Grit Chambers and Screens; Tanks; Filters; Disinfection; Chemical Precipitation and Electrolytic Treatment; Trade Wastes; Testing Stations; Making Tests; Records of Plant Operation; Index.

DESIGN OF STEEL BRIDGES:

Theory and Practice, for the Use of Civil Engineers and Students. By F. C. Kunz, M. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., illus., 24 + 472 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $5.00.

This book, the preface states, is the first of four, each complete in itself, which are to include data collected by the author during thirty years of professional practice. The aim of this volume, it is stated, is to give the bridge engineer numerical examples and results of the best modern practice in designing and estimating steel bridges, and to serve him as a guide and aid in the calculation of stresses, sections, weights, etc. The text and methods of calculation, both graphic and algebraic, have been condensed, it is said, and fully cross-referenced to the tables and plates, in order to make all parts of the book readily accessible. Methods using influence lines have been preferred for graphic calculations, it is stated, and the simplest algebraic methods have been used. Arches without hinges and suspension bridges are not discussed in this volume because of their limited use, but the treatment of the various types of two-hinged arches is full, it is said, and intended for immediate use in design. The influence of lateral forces on the stresses in the main trusses of arch and cantilever bridges has been given special attention, the author states, on account of the increased use of alloy steels for span lengths. In Chapters XV and XVI, descriptions of existing and of a few completely designed long-span bridges are given, including analyzed weights, specifications of materials, loads and permissible unit stresses. Appended to several chapters is additional information on the subjects discussed, which, it is said, has been carefully compiled from the author's library and notes. The Appendix includes original data on foundation pressures of bridges and other structural work, and other useful tables, as well as Part First, Design, of the General Specifications for Steel Railway Bridges of the American Railway Engineering Association. The author has also added fifty-two plates which are intended to serve as a guide to the designer from the live and dead load assumptions to the last rivet spacing, without reference to the text. The Contents are: External Forces; General About Reactions and Influence Lines; Moments and Shears in Simple Spans; Stresses in Simple Trusses; Stresses in Bracing of Simple Spans; Types of Bridges and Principal Dimensions; Design of Floor; Beam and Plate Girder Bridges; Simple Truss Bridges; Skew Bridges and Bridges on Curves; Weights of Simple Span Bridges; Viaducts; Elevated Railroads; Movable Bridges and Turntables; Arch Bridges; Long Span Bridges in General and Examples; Cantilever Bridges; Additional Information on Long Span Bridges; Appendix; Index.

PLAIN AND REINFORCED CONCRETE ARCHES.

By J. Melan. Authorized Translation by D. B. Steinman, Jun. Am. Soc. C. E. Cloth, 9¼ x 6 in., illus., 10 + 161 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $2.00.

As stated in the preface, this book is a translation of Professor Melan's article published in von Emperger's "Handbuch für Eisenbetonbau," the second revised edition (1912) of which is represented in this translation. Professor Melan's work is said to be one of the most thorough treatments of reinforced concrete arches in any language. In it the fundamental principles of arches is briefly discussed and a simple comprehensive treatment of stresses is given, together with analytic and graphic methods for the design of all types of concrete arches occurring in practice, as well as the effects of temperature, yielding abutments, and non-vertical loads. The translator states that the following special features should recommend this book to American engineers: Simple approximations and short cuts useful for preliminary and less exacting designs; easily applied formulas for determining in advance the best curve for an arch and the required dimensions and reinforcements; illustrations of the various methods described by numerical examples from actual practice; full explanations and demonstrations of the principles of the Melan system of arch construction; complete computations of analytic and

graphic stresses of two notable examples of reinforced concrete arches; and a chart for proportioning or investigating concrete sections with single or double reinforcement. It is stated that the notation has been modified in this translation, that an index of symbols is included and that all numerical examples and data have been converted from metric to English units. There is also a short bibliography of the subject. The Contents are: The Forces Acting on an Arch; The Internal Stresses in an Arch; The Three-Hinged Arch; The Hingeless Arch; Effects of Temperature and Displacement of the Abutments; Non-Vertical Loads; Approximate Methods for the Hingeless Arch; Two-Hinged and One-Hinged Arches; Arches with Elastic Abutments; Arches Continuous over Several Spans; Determining the Best Curve for Any Arch; Determining the Thickness for Any Arch; Proportioning Reinforced Concrete Arches; Calculating the Stresses in an Arch; Examples; References; Key to Plate I; Index.

EXAMINATION OF LUBRICATING OILS.

By Thomas B. Stillman. Cloth, 9 x 6 in., illus., 125 pp. Easton, Pa., The Chemical Publishing Co.; London, Williams & Norgate, 1914. $1.25.

The rapid advancement in the construction of machinery of various kinds requires, it is stated, corresponding varieties of oils for lubricating purposes and has also complicated the methods of testing such oils. This is particularly true, the author states, of the automobile and of cold storage specifications for lubricating oils. After describing the different methods and apparatus used in the various tests, the author gives specifications for different lubricating oils and fuel oils, together with analyses of such oils, tables of the approximate composition of the crude oils of the United States, of products obtained from Pennsylvania crude petroleum when distilled destructively, and for comparison of Centigrade and Fahrenheit degrees, as well as wholesale prices current of the various substances used in lubrication. The author calls particular attention to the chapter on "Apparatus for the Examination and Study of the Behavior of Valve and Cylinder Oils and Other Petroleum and Lubricating Oils in Saturated and Superheated Steam, Carbon Dioxide and Other Gases," by P. H. Conradson, Chief Chemist of the Galena-Signal Oil Company, of Franklin, Pa., which he recommends as an experiment of great value on this subject. There is also a short bibliography of the subject at the end of the book. A partial list of Contents is: Specific Gravity; Cold Test; Viscosity; Iodine Absorption; Flush and Fire Test; Acidity; Manmene's Test; Color Reactions of Oils with Nitric and Sulphuric Acids; Separations of Material Oil from a Vegetable or Animal Oil; Gumming Test; Sulphur Test; Test for Water; Gasoline Test; Microscopical Examination; Carbon Residue Test; Fixed Carbon in Oil; etc., etc.

THE "MECHANICAL WORLD" POCKET DIARY AND YEAR-BOOK FOR 1915:

A Collection of Useful Engineering Notes, Rules, Tables and Data. Cloth, 6¼ x 4 in., illus., 439 pp. Manchester and London, England, Emmott & Company, Limited; Baltimore, Md., The Norman, Remington Co., 1915. 50 cents. (Donated by The Norman, Remington Co.)

The preface states that this, the 1915, edition of the Handbook on the mechanical means of transmitting power, has been thoroughly revised and that many new illustrations and features containing matter of value to the engineer, draftsman, etc., have been added. The section on Toothed Gearing has been rewritten and enlarged and now embodies, it is said, much practical data on this subject and also on Gear Cutting. A section on Structural Steel and Iron Work has been substituted for that formerly devoted to Beams and Girders. The section on the Gas Engine has been revised and extended, and new sections on Limit Gauges, Strength of Flat Plates, etc., have been added, together with new tables on Cost of Power, Helix Angles, Morse Tapers, Proportions of T-Slots, etc. A partial list of Contents is: Index; Steam and the Steam Engine; Steam Turbines; Steam Boilers; Gas Engines; Oil Engines; Suction Gas Producers; Structural Iron and Steel Work; Shafting; Toothed Gearing; Indexing on the Universal Milling Machine; Notes on Grinding; etc., etc.

CENTRIFUGAL PUMPS.

By R. L. Daugherty. Cloth, 9½ x 6¼ in., illus., 9 + 192 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $2.00.

The extensive use and increasing popularity of centrifugal pumping machinery make it necessary, it is stated, for many engineers to familiarize themselves with all phases of the subject. In this book, the author has endeavored, it is said, to

illustrate and explain all the essential features of construction, installation, and operation of the various types of modern centrifugal pumps, as well as their suitability to various services. He also discusses the general theory of the centrifugal pump, its characteristics, factors of efficiency, etc., and presents a comparison of it and the displacement pump, together with the various laws and factors leading to a better appreciation of such pumps and a better means of selecting proper combinations. The subject-matter, it is stated, is based on the author's study of the performances and analyses of tests of turbines and volute centrifugal pumps made by various manufacturers, and although it is not intended as a book on design, it is believed that a thorough study of its contents will be of value to prospective designers, the methods of design of centrifugal pumps being also outlined. It has been written, it is said, to serve the needs of the practicing engineer, but problems and questions have been inserted at the end of each chapter, which it is hoped will make it of equal value as a textbook. The Chapter headings are: Introduction; Description; Installation and Operation; General Theory; Theory of Centrifugal Pumps; Characteristics; Disk Friction; Factors Affecting Efficiency; Centrifugal Pumps *vs.* Displacement Pumps; Comparison of Types of Centrifugal Pumps; General Laws and Factors; Pump Testing; Costs; Rotary and Screw Pumps; Applications of Centrifugal Pumps; Design of a Centrifugal Pump; Appendix A, Test Data; Appendix B, Review Questions; Appendix C, Table of ¾ Powers of Numbers; Index.

HANCOCK'S APPLIED MECHANICS FOR ENGINEERS.

Revised and Rewritten by N. C. Riggs. Cloth, 7½ x 5¼ in., illus., 13 + 441 pp. New York, The Macmillan Company, 1915. $2.40.

In the author's preface to the first edition, it is stated that the subject-matter includes much that is new in the applications of mechanical principles and that the chapters on Couples, Moment of Inertia, Center of Gravity, Work and Energy, and Friction and Impact, are more complete in theory and application than those of any other American textbook on the subject. The volume is intended, it is stated, as a textbook for engineering students of the Junior year, but it is hoped that it may also prove helpful to engineers. The student, it is said, finds much difficulty in applying theory to practical problems, and, therefore, each new principle developed in the text has been followed by problems illustrating such principle. Frequent reference to original sources are also given. In the revised edition, it is said, one important change has been the much larger use of graphical methods, the graphical and analytical methods being developed simultaneously and many of the problems being solved by both methods. The original subject-matter and order of arrangement of the text have been retained, it is stated, but much that is new has been introduced, particularly in the construction of stress diagrams for trusses, the application of equilibrium polygons to centers of gravity of plane areas, to weighted strings and linkages, and to bending moment diagrams, etc. About two hundred new problems have also been added for solution in connection with the principles discussed. The Contents are: Definitions; Concurrent Forces; Parallel Forces; Center of Gravity; Couples; Non-Concurrent Forces; Moment of Inertia; Flexible Cords; Motion in a Straight Line; Curvilinear Motion; Rotary Motion; Work and Energy; Friction; Dynamics of Rigid Bodies; Impact; Appendix I, Hyperbolic Functions; Appendix II, Logarithms of Numbers; Appendix III, Trigonometric Functions; Appendix IV, Conversion Tables; Index.

THE "MECHANICAL WORLD" ELECTRICAL POCKET BOOK FOR 1915:

A Collection of Electrical Engineering Notes, Rules, Tables and Data. Cloth, 6¼ x 4¼ in., illus., 303 pp. Manchester and London, England, Emmott & Company, Limited; Baltimore, Md., The Norman, Remington Co., 1915. 50 cents. (Donated by The Norman, Remington Co.)

This Pocket Book, it is stated, is a companion volume to the "Mechanical World" Pocket Diary and Year Book, and includes a varied collection of electrical data prepared especially for the use of those in charge of electrical plants and machinery. The subject-matter, it is said, has been thoroughly revised and extended and several new sections have been added on Electric Circuits and Switching, Synchronizing, Phasing Out, Alternating Current Generators, Motors, etc. A partial list of Contents is: Index; Electrical Units; Resistance; Electrolysis; Magnets; Magnetic Circuit and Magnetic Materials; Continuous Current Dynamos and Motors; Alternate Current Systems; Alternators; Alternating Current Motors; Alternating Current Generator and Motor Troubles; Transformation of Currents; Motor Starters; Supply Factors; Accumulator Notes; Transmission Conductors and Cables; Conduit System of Wiring; etc., etc.

ELEMENTARY ELECTRICITY AND MAGNETISM:

A Text-Book for Colleges and Technical Schools. By Wm. S. Franklin and Barry Macnutt. Cloth, 7¾ x 5 in., illus., 8 + 174 pp. New York, The Macmillan Company; London, Macmillan & Co., Ltd., 1914. $1.25.

The preface states that the study of electricity and magnetism, as represented in this book, may be properly called electro-mechanics. The subject-matter, as stated in the title, is elementary in character, and is said to show the variant co-relations involved in electric currents, electromotive forces, and the magnetic and electric fields. At the end of each chapter problems in the subjects discussed in that chapter are given. The Chapter headings are: Effects of the Electric Current; Magnetism; Chemical Effect of the Electric Current; The Heating Effect of the Electric Current; Induced Electromotive Force; Electric Charge and the Condenser; Index.

HANDBOOK ON OVERHEAD LINE CONSTRUCTION.

Compiled by the Sub-Committee on Overhead Line Construction, National Electric Light Association. Presented at the Thirty-Seventh Convention, Held at Philadelphia, Pennsylvania, June 1-5, 1914. Roan, 6¾ x 4½ in., illus., 10 + 819 pp. Philadelphia, National Electric Light Association, 1914.

The purpose of this Handbook, as stated in the preface, is the presentation in one volume of descriptions of the methods and materials used in overhead line construction and a tabulation of the necessary formulas for the electrical and mechanical solutions of various transmission and distribution problems. In the discussion of apparatus used for the purpose, efforts have been made, it is said, to describe the various types at present on the market. The Sub-Committee states that this work is not to be considered as a Handbook of rules and regulations, and that no attempt has been made to create standards or write specifications; it is rather a collection of useful information which should prove of material assistance to those engaged in the construction or maintenance of overhead lines for light or power purposes. The Contents are: An Abridged Dictionary of Electrical Words, Terms, and Phrases; Tables; Distribution and Transmission Line Supports; Conductors and Wire Tables; Cross-Arms, Pins and Pole Line Hardware; Insulators; Transformers and Induction Regulators; Systems of Distribution and Transmission; Electrical Calculations; Mechanical Calculations of Transmission and Distribution Lines; Preservative Treatment of Poles and Cross-Arms; Primary and Secondary Line Construction; Meteorological Data; General Data, and Rules for Resuscitation from Electric Shock; Index.

FIELD PRACTICE: AN INSPECTION MANUAL

For Property Owners, Fire Departments, and Inspection Offices, Covering Common Fire Hazards and Their Safeguarding and Fire Protection and Upkeep. 1914 Edition. Roan, 7 x 4¾ in., 199 pp. Boston, National Fire Protection Association, 1914. $1.50.

As stated in the title, this Manual covers the more essential features of the common fire hazards and their safeguarding, the functional principles, as well as the maintenance and upkeep, of fire protection appliances, and the means of fire prevention. It embodies, it is said, in non-technical language, the latest ideas of the leading American fire prevention engineers, and is intended to serve as a guide in these matters for the layman and the inspector. The Contents are: I, Common Fire Hazards and Their Safeguarding: Lighting Hazards; Heating Hazards; Power Hazards; Chemicals, Paints and Oils; Spontaneous Ignition and Dust Explosions; Care and Maintenance; Chimneys and Flues; Dwelling House Hazards; II, Fire Protection and Upkeep (Automatic and Manual): Automatic Sprinkler Installations; Fire Protection in General; Index.

MASONRY:

A Short Text-Book on Masonry Construction, Including Descriptions of the Materials Used, Their Preparation and Arrangement in Structures. By Malverd A. Howe, M. Am. Soc. C. E. Cloth, 9¼ x 6 in., illus., 9 + 160 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $1.50.

The object of this book, it is stated, has been to furnish a concise treatment of masonry construction for use in a short course which includes concrete con-

struction or in a longer course in conjunction with a manual on concrete. Only modern methods of construction are described, it is stated, and modern tools and machinery illustrated. In several chapters, definitions of terms commonly used in connection with the subject discussed are included, and the specifications in Part III are said to give the best modern methods of selecting materials and their use in structures. A detailed bibliography of the subject is also given. The Contents are: Part I, The Materials; Natural Building Stones; Artificial Building Materials. Part II, Masonry: Stone Masonry; Brick and Hollow-Tile Masonry; Concrete Masonry. Part III, Railroad Masonry, Brick, Cement, etc.; References; Index.

THE MECHANICAL PROPERTIES OF WOOD

Including a Discussion of the Factors Affecting the Mechanical Properties, and Methods of Timber Testing. By Samuel J. Record. Cloth, 9¼ x 6 in., illus., 11 + 165 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1914. $1.75.

This book, the preface states, was written primarily for students of forestry to whom a knowledge of the technical properties of wood is essential, but all unnecessary technical terms and descriptions have been avoided, it is said, in an endeavor to make the work available to every one interested in wood. The subject-matter is divided into three parts. In Part I, the author discusses the relation of wood material to stresses and strains and includes numerous tables showing the various strength values of many of the more important American woods. Part II deals, it is said, with the factors affecting the mechanical properties of wood, and the author states that, in this connection, he has attempted to answer such questions as the effect on the quality of wood of rate of growth, season of cutting, heartwood and sapwood, locality of growth, weight, water content, steaming, and defects. In Part III, methods of timber testing, as well as the various machines used in such testing, are described. These methods are stated to be mostly those followed by the United States Forest Service. The Appendix contains a sample working plan followed by the Forest Service in extensive investigations covering the mechanical properties of woods grown in the United States, together with tables of strength values of both green and air-seasoned timbers for structural purposes. There is a detailed bibliography of the important publications and articles on the mechanical properties of wood and timber testing. The Contents are: Part I, The Mechanical Properties of Wood; Part II, The Factors Affecting the Mechanical Properties of Wood; Part III, Timber Testing; Appendix; Bibliography; Index.

PRACTICAL TALKS ON FARM ENGINEERING:

A Simple Explanation of Many Everyday Problems in Farm Engineering and Farm Mechanics Written in a Readable Style for the Practical Farmer. By R. P. Clarkson. Cloth, 7¾ x 5¼ in., illus., 15 + 223 pp. Garden City, N. Y., Doubleday, Page & Company, 1915. $1.00.

The author's aim in this book, it is stated, is to present, in an interesting and popular manner, material gleaned from years of experience spent in aiding and advising farmers on the subject. The subject-matter is said to be a selection from articles, published by the author in prominent farm journals in the United States and Canada, in answer to questions relating to engineering subjects with which the farmer has to deal, such as choice of materials for buildings, roads, walks, and fences, and their construction, water supply, land drainage, sewerage and sewage disposal, power, etc. The text, it is stated, has been thoroughly revised and expanded, and many new illustrations have been added. The Chapter headings are: Farm Buildings and Building Materials; Farm Water Supply and Sewage Disposal; Farm Power; Drainage and Irrigation; Miscellaneous Engineering Talks; Useful Tables for Engineering Calculations; Index.

ELEMENTS OF FORESTRY.

By Frederick Franklin Moon and Nelson Courtlandt Brown. Cloth, 8¼ x 5½ in., illus., 17 + 392 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited; Montreal, Renouf Pub. Co., 1914. $2.00.

The educational awakening that is taking place along forestry lines in the United States has been so remarkable, it is stated, that the authors felt that a textbook, broad in scope and containing general information on all phases of the

subject, was needed for use in agricultural schools and colleges. They have endeavored, therefore, in this volume to supply such need. Their chief object has been, it is said, to gather data on the subject from sources not readily available and to present them in a form easily grasped by the student. An especial effort has been made, it is said, to make the book useful to teachers without a forestry degree who are engaged in agricultural work, and it is hoped that the bibliographies at the end of each chapter will prove of particular interest to them. Many details in regard to the forest regions, which can be supplied by individual instructors, have been omitted, it is stated, as well as a chapter on Dendrology which is thought to be too large a subject for a general textbook. The Appendix contains tables of log rules, volume, yield, rate of growth, etc., sample tally sheets for use in estimating and stem analysis, etc., and a Glossary of terms used in Forestry is also included. The Chapter headings are: Introduction; The Tree; Silvics; Silvicultural Systems of Management; Improvement Cutting; Artificial Regeneration; Forest Protection; Forest Mensuration; Lumbering; Wood Utilization; Wood Technology; Wood Preservation; Forest Economics; Forest Finance; Regional Studies; Northern Forest; Southern Pines; Central Hardwoods; Prairie or Fringe Forest; Northern Rocky Mountain Forest; Southern Rocky Mountain Forest; Pacific Coast Forest; Appendix; Glossary; Index.

HISTORY OF RENSSELAER POLYTECHNIC INSTITUTE, 1824-1914.

By Palmer C. Ricketts, M. Am. Soc. C. E. Cloth, 8¾ x 6 in., illus., 12 + 269 pp. New York, John Wiley and Sons; London, Chapman & Hall, Limited, 1914. $2.50. (Donated by the Author.)

The first edition of this book was published in 1895, and in the preface to that edition, the author states that because the official publications relating to the Institute from the time of its founding had become rare, he had determined to publish a short history of the school which should consist largely of a description of the development of its curriculums. The Institute was founded, it is stated, at Troy, N. Y., in 1824, by Stephen Van Rensselaer, of Albany, N. Y., under the name of the Rensselaer School, for the purpose of teaching the "application of science to the common purposes of man" and was the third school of its kind in any English-speaking country and the first which has had a continuous existence. As stated, this volume contains detailed information of the Institute, its founders, different courses, equipment, methods, etc., including a history of its curriculums, from its founding in 1824 to 1914. The Contents are: The Foundation of the School; Stephen Van Rensselaer and Amos Eaton; Act of Incorporation and Early By-Laws; Methods of Instruction, Preparation Branch Established; Name Changed to Rensselaer Institute, Removal to the Van der Heyden Mansion; Establishment of the Department of Civil Engineering; Reorganization of the School: The Rensselaer Polytechnic Institute; Destruction by Fires, More Land and New Buildings, Athletics; Russell Sage, Mechanical and Electrical Courses, Graduate Courses, '87 Gymnasium; Alumni and Student Organizations, Publications, Statistics of Graduates; Present-Day Equipment and Methods, Statistics of Students; Bibliography; Appendices; Index.

CIVIL ENGINEERING SPECIFICATIONS AND CONTRACTS.

By Richard I. D. Ashbridge, M. Am. Soc. C. E. Cloth, 8½ x 5½ in., illus., 186 pp. Chicago, American Technical Society, 1914. $1.00. (Donated by the Author.)

In an engineering work, the author states, it is important that the interested parties consider what is to be done, how it is to be done, and what it is going to cost. To this end the engineer, it is said, must set forth his ideas by drawings and written descriptions or specifications, make estimates of cost under various methods of construction, prepare the form of bid or proposal, and, later, must adopt a form of articles of agreement and fix a bond which, with the specifications, drawings and proposal, form the contract. These various steps are treated in this book, it is stated, in the order of their importance in the contract from the engineer's point of view. The author has endeavored, it is said, to develop a logical system of preparing specifications and to present a method of avoiding mistakes and omissions that are common to such work. All types of specifications for railroad work, bridges, culverts, excavations, fills, tunnels and roadbeds, and country and city paving, have been covered, it is said, and proposals, agreements, and contract forms have also been discussed, together with points to be considered and avoided in drawing up a set of specifications and in the proper drafting of a contract. The Contents are: Introduction; General Instructions; General Provisions; Typical Illustrative Specifications; Contract or Articles of Agreement; Proposals; Advertisement; Practice in Specification and Contract Writing; Index.

ENGINEERING OFFICE SYSTEMS AND METHODS,

Together with Schedules and Instructions for the Collection of Preliminary Data for Engineering Projects; Sampling, Inspecting and Testing Engineering Materials; Conducting Domestic and Export Shipping Operations, etc. By John P. Davies, Assoc. M. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., illus., 16 + 544 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $5.00 (Donated by the Author.)

The subject-matter contained in this book had its inception, the preface states, in a collection of such "reminders" as is commonly used in engineering offices for checking drawings and specifications, together with matter relating to engineering office routine, extracts from articles in periodicals, catalogues, and engineering books relating to the general subject of engineering office systems. As stated in the secondary title, the author has discussed the collection of preliminary data for engineering projects, including foundation testing, stream gauging, engineering designs and drawings, specification writing, quotations on standard materials for plant, machinery, materials of construction, etc., purchasing methods, cost keeping and estimating, progress charts, scheduling systems, etc., indexing and filing systems, drawing-room systems and methods, abbreviations, fees, contract forms, etc. The Chapter headings are: Collection of Preliminary Data for Engineering Projects; Designing and Drafting Systems; Specifications for Engineering Material; Reminders for Obtaining Quotations on Standard Material; Purchasing-Office Methods and Forms; Cost Keeping and Estimating; Sampling, Inspecting and Testing Engineering Material; Domestic Shipping; Export Shipping; Progress Charts, Scheduling Systems, etc.; Indexing and Filing Systems, etc., for the Engineering Office; Drawing-Office Systems and Methods; Miscellany; Index.

CONCRETE STONE MANUFACTURE.

By Harvey Whipple. Roan, 7 x 4¼ in., illus., 255 pp. Detroit, Concrete-Cement Age Publishing Co., 1915. $1.00.

The development of factory-made concrete units, the preface states, has not kept pace with that of field-made concrete in the mass, and this is due, the author asserts, to the fact that many individual enterprises have not had adequate managerial and mechanical equipment. A proper development of concrete stone manufacture depends, it is said, on a study of the market, on skillful workmanship, economical operation, adequate equipment, and competent management, and these objectives, it is stated, are considered in detail in this book which it is hoped will lend impetus and direction to an important industrial growth. In Chapter 10, the author has given descriptions of the layout and equipment of eleven commercially operated factories engaged in the industry, together with plans, specifications, and costs, which chapter is intended as an aid to the manufacturer who is planning to erect and equip a new plant. The Contents are: The Development of Concrete Building Units; Location, Equipment, Layout; Materials, Mixtures, Manipulation; Curing; Special Molds and Patterns; Surfaces; Shop Records and Cost Keeping; Building Regulations, Tests, Specifications; Selling the Products; Examples of Layout and Operation; Index.

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O'HEARN, JOHN LYNCH. Cons. Engr., 811 North Bishop, Dallas, Tex.

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STEPHENS, ALLEN WHITMORE. Engr., Turner Constr. Co., 11 Broadway (Res., 120 West 57th St.), New York City.

STEVENS, LEIGH E. Sioux Falls, S. Dak.

STODDARD, RAYMOND FRENCH. Cons. Engr., The Gulf, Milford, Conn.

SWATY, DAVID YOUNGS. Engr. with Great Lakes Dredge & Dock Co., 1630 Williamson Bldg., Cleveland, Ohio.

TALLMAN, LEROY. Supt., Flinn-O'Rourke Co., Inc., East River Tunnels, Clark and Furman Sts., Brooklyn, N. Y.

THANHEISER, CHARLES AUGUST. Supt., M., K. & T. Ry., Smithville, Tex.

TINKHAM, RALPH RUSSELL. Supt., 16th Dist., U. S. Lighthouse Service, Ketchikan, Alaska.

TROTT, DAVID CROOKER. Supt. of Constr., U. S. Public Bldgs., Bozeman, Mont.

TUDBURY, WARREN CHAMBERLAIN. Civ. and Landscape Engr., 913 South Figueroa St., Los Angeles, Cal.

TURNER, AUGUSTUS MIESSE. Engr., M. of W., Peoria and East. Div., C., C., C. & St. L. Ry., Indianapolis, Ind.

TURNEY, OMAR ASA. 357 North Fourth Ave., Phœnix, Ariz.

TYLER, ROY DEXTER. Cons. Engr., 3039 Wilson Ave., Chicago, Ill.

VANDERVOORT, BENJAMIN FRANKLIN. 587 West 178th St., New York City.

VAN RENSSELAER, ALLEN. 1200 Pine St., San Francisco, Cal.

WALTER, THOMAS ROBERT. Civ. and Architectural Engr., 915 South Park St., Kalamazoo, Mich.

WIGTON, CHARLES BENSON. Contr. Engr., Levering & Garrigues Co., 552 West 23d St., New York City.

WILSON, WILLIAM RENFREW. Care, Hongkong Shanghai Bank, London, England.

WORLEY, ALBERT HARRISON. With The Interstate Commerce Comm., 3622 Garfield Ave., Kansas City, Mo.

ASSOCIATES

BELZNER, THEODORE. Insp. in Chg., 155th Street and Riverside Drive Viaducts, Dept. of Bridges (Res., 606 West 135th St.), New York City.

ASSOCIATES (*Continued*)

BYERS, BENJAMIN BUTLER FRANKLIN. 1214 Murdock Ave., Parkersburg, W. Va.

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PINCHOT, GIFFORD. 1214 Real Estate Trust Bldg., Philadelphia, Pa.

PULLAR, HAROLD BEGGS. Gen. Mgr., The Pioneer Asphalt Co., Lawrenceville, Ill.

SOMMER, ALBERT. Mgr., Deutsche Trinidad Asphalt Gesellschaft, Münchnerplatz 14, Dresden, Germany.

JUNIORS

ACKHART, ANDREW LEWIS. Highland, N. Y.

ANDERSON, JOHN HENNING. Asst. Engr., Universal Portland Cement Co., 532 Frick Bldg., Pittsburgh, Pa.

BAKER, HAROLD WALLACE. 208 Main St., Oneida, N. Y.

BENDEL, JACOB. Junior Engr., Public Service Comm., First Dist. (Res., 943 Whitlock Ave.), New York City.

BITHER, TOM ALLEN. 1630 Josephine St., Berkeley, Cal.

BOYYER, WILLIAM BLAIR. Asst. City Engr., 760 Eighth Ave., San Francisco, Cal.

BRADSTREET, HERBERT NEAL. Care, Surveyor-General of Arizona, Phœnix, Ariz.

BRINKERHOFF, GEORGE LOCKWOOD. 1279 Cole St., Columbus, Ohio.

CASPARI, FREDERICK WILLIAM. Res. Engr., State Dept. of Health, 16 West Saratoga St., Baltimore, Md.

DIMMLER, CHARLES LOUIS. Asst. Engr., Div. of Works, Panama-Pacific International Exposition, 1511 Masonic Ave., San Francisco, Cal.

EBERLY, VIRGIL ALLEN. Care, Interstate Commerce Comm., 9th Floor, Karpen Bldg., Chicago, Ill.

GILKISON, GORDON MERCER. 34 Arkledun Ave., Hamilton, Ont., Canada.

KELLY, JOHN ARTHUR. 900 South 10th St., Philadelphia, Pa.

KING, TAO. Asst. Engr., Nanking-Hunan Ry., Care, Post-Office, Nanchang, Kiangsi, China.

KOCH, OTTO HERMAN SIEGFRIED. 1134 Simpson St., New York City.

LUCCHETTI-OTERO, ANTONIO SEBASTIAN. Care, Bureau of Public Works, San Juan, Porto Rico.

MAYO, GEORGE. 1921 Nineteenth St., N. W., Washington, D. C.

MORGAN, JOSEPH HOLLOWAY. Junior Engr., U. S. Geological Survey, Water Resources Branch, 328 Custom House, San Francisco, Cal.

MURPHY, ALVIN RUSH. Fountain City, Tenn.

PICKFORD, EDMUND JOHN. Care, Edmund D. Pickford, P. O. Box 4242, Johannesburg, Transvaal, South Africa.

PIPER, HARRY PAUL, JR. Mgr., Cost Dept., S. S. Sanford & Sons, Inc., 5½ Grant Ave., Amsterdam, N. Y.

RICE, ROGER CUSHING. Care, U. S. Geological Survey, Kapiolani Bldg., Honolulu, Hawaii.

JUNIORS (*Continued*)

ROSSI, IRVING. Structural Draftsman, Milliken Bros., Inc., 215 Fairview Ave., Jersey City, N. J.

SCHROEDER, SEATON, JR. Engr., M. of W., Charleston Interurban R. R., Charleston, W. Va.

SCHUYLER, WALTER WESLEY. Care, United Fruit Co., Bocas del Toro, Panama.

SHAPLEIGH, CHARLES HENRY. Asst. Engr. to the Asst. to the Pres., N. O. & N. E. R. R., Q. & C. Bldg., New Orleans, La.

SHAW, GUY RAY. Victoria Hotel, Des Moines, Iowa.

SMITH, WILLIAM DURKEE. Structural Draftsman, Port of Seattle Comm., 112 North 51st St., Seattle, Wash.

WARING, FREDERICK HOLMAN. Filtration Chemist, Miraflores Filtration Plant, Corozal, Canal Zone, Panama.

RESIGNATIONS

MEMBERS	Date of Resignation.
COMPTON, CHARLES SUMNER	Dec. 31, 1914
IVES, ARTHUR STANLEY	Dec. 31, 1914
ASSOCIATE MEMBERS	
BABÉ, JOSEPH MANUEL	Dec. 31, 1914
DUFFEE, LOUIS WARREN	Dec. 31, 1914
KORSMO, AMUND MARIUS	Dec. 31, 1914
SESSER, JOHN CORNELIOUS	Dec. 31, 1914
JUNIORS	
EDMUNDSON, HAROLD BOWEN	Dec. 31, 1914
MALMROS, NILS LORENTZ ALFRED	Dec. 31, 1914
OTTOSEN, PETER HILL	Dec. 31, 1914

DEATHS

COOKE, SAINT GEORGE HENRY. Elected Associate Member, January 5th, 1909; died January 12th, 1915.

CROWELL, FOSTER. Elected Member, December 1st, 1880; died March 29th, 1915.

HAMBLETON, FRANCIS HENRY. Elected Member, March 5th, 1873; died March 19th, 1915.

HUNTER, WILLIAM. Elected Member, June 5th, 1895; died April 2d, 1915.

JOHNSON, CHAPMAN LOVE. Elected Member, October 7th, 1903; died March 11th, 1915.

MCFARLAND, WALTER ASHFIELD. Elected Member, May 3d, 1910; died March 17th, 1915.

MOORE, WILLIAM EDWIN. Elected Member, February 7th, 1906; died January 24th, 1915.

DEATHS (*Continued*)

MORRIS, HENRY GURNEY. Elected Member, December 4th, 1867; date of death unknown.

RICH, ISAAC. Elected Member, May 6th, 1903; died March 11th, 1915.

RUSSELL, WILLIAM GARDNER. Elected Member, October 4th, 1905; died March 5th, 1915.

Total Membership of the Society, April 8th, 1915, 7 737.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(March 2d to April 3d, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Steel and Iron,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71a) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 20c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Iron Tradesman,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung,* Leipzig, Germany.

LIST OF ARTICLES

Bridges.

Construction of the Fallsway Viaduct, Baltimore, Md.* Louis R. Gons. (67) Mar.
Renewal of Maribyrnong (Saltwater River Bridge), Victorian State Railways.* F. K. Esling. (From the *Commonwealth Engineer.*) (21) Mar.
Billings Bridge over Rideau River, Ottawa.* (96) Mar. 4.
Concrete Arch Bridge at Saskatoon.* (13) Mar. 4; (96) Mar. 18.
The Erection Traveler, New Quebec Bridge.* (13) Mar. 4.
New Swing-Bridge, Whitby.* W. Noble Twelvetrees. (11) Serial beginning Mar. 5.
Viaduct Construction on the Kansas City Terminal.* A. R. Eitzen. (15) Mar. 5.
Wood Block Replaces Plank as Steel Bridge Flooring. Edward Stingel. (14) Mar. 6.
Replacing a Large Truss Bridge by Lateral Movement. J. C. Bland and John Miller. (Paper read before the Am. Assoc. for the Advancement of Science.) (15) Mar. 12.

* Illustrated.

Bridges—(Continued).

Coating Disintegrated Stone Abutments with Concrete; Repair Work under Traffic of Bridge of Chicago & Western Indiana Railroad over Chicago Drainage Canal.* (14) Mar. 13.
Dismantling Truss Spans and Erecting Plate Girders Without False Work, W. & L. E. R. R.* E. V. Smith. (18) Mar. 13.
New Concrete Arch Viaduct of the Philadelphia, Baltimore & Washington R. R. over Gwynns Falls, Baltimore, Md.* (18) Mar. 13.
Pennsylvania's Concrete Bridge over the Susquehanna.* (14) Mar. 13.
Rolling Lift Bridges for the Delray Connecting R. R.* (18) Mar. 13.
Some Bridge Work of the C. M. & St. P. Ry.* (18) Mar. 13.
Substructure for the Jackson St. Bridge over the Chicago River.* (13) Mar. 18.
The Development of the Sudan.* (12) Mar. 19.
Notable Structures on the Spokane-Ayer Cut-off.* (15) Mar. 19.
Design of the Reinforced Concrete Cantilever Bridge on Runnymede Ave., Cleveland, Ohio.* (86) Serial beginning Mar. 24.
A Balanced Cantilever Reinforced-Concrete Bridge.* Henry H. Quimby. (13) Mar. 25.
Congress St. Bridge across the Hudson River at Troy, N. Y.; Structural Features.* Henry W. Hodge. (13) Mar. 25.
Jacking up a Concrete Arch over a Settling Pier.* W. P. Darwin. (13) Mar. 25.
Record Set for Weight of Steel Erected in One Day.* (14) Mar. 27.
The Economic Design of Culverts for Various Depths of Fills.* P. K. Sheidler. (Paper read before the Ohio Eng. Soc.) (86) Mar. 31.
An Economical Bridge-Pier Foundation.* Milo K. Temple. (13) Apr. 1.
Low Water Bridges over Torrential Streams, Bexar Co., Texas.* Terrell Bartlett. (13) Apr. 1.
Wind Stresses in Skew Bridges.* J. P. J. Williams. (13) Apr. 1.
Substructure Conditions Fix Design of Chicago & North Western Bridge at Pekin.* C. F. Dalston. (14) Apr. 3.
Le Pont de Sara, sur le Gange.* (33) Mar. 6.
Pont en Béton Armé, sur l'Aar, à Olten (Suisse).* (33) Mar. 20.
Neues Verfahren zur raschen Ermittelung der Biegungsmomente in eingespannten Gewölben nebst Pfeilern und Widerlagern. R. Färber. (51) Serial beginning Sup. No. 5.
Winddruck bei Brücken. Heinrich Saller. (40) Jan. 2.

Electrical.

Induction Regulators.* G. H. Eardley-Wilmot. (73) Feb. 19.
Alternating Current Reverse Relays.* C. C. Garrard. (73) Serial beginning Feb. 26.
The Electricity Supply to Port Glasgow from Greenock.* (26) Feb. 26.
Fractional Horse-Power Motor Load.* Bernard Lester. (42) Mar.
Methods, Data, and New Apparatus for Measuring Electrical Conductivity Above 1500° C. of Vapors at Normal Pressure.* Edwin F. Northrup. (3) Mar.
Paints to Prevent Electrolysis in Concrete Structures.* Henry A. Gardner. (3) Mar.
Some Troubles Encountered in the Operation of Carbon Brushes in Direct-Current Generators and Motors.* E. H. Martindale. (42) Mar.
Train Dispatching by Wireless. L. B. Foley. (65) Mar.
Transcontinental Telephone Service.* P. C. Staples. (98) Mar.
Future Operation of Long Electric Transmission Lines.* R. A. Philip. (13) Mar. 4.
The Kolster Decremeter.* (73) Mar. 5.
Projected New Power Station for Manchester. (12) Mar. 5.
The Ultraudion Detector for Undamped Waves.* Lee De Forest. (73) Mar. 5.
Records of Radio Time Signals.* C. W. Waggoner. (Paper read before the Am. Physical Soc.) (19) Mar. 6.
The Selective Time Element of Relays.* Paul MacGahan. (27) Mar. 6.
The Beck Searchlight.* (11) Mar. 12.
Underground Wires on the Panama Railroad.* (15) Mar. 12.
Searchlight Projectors.* C. W. Denny. (26) Serial beginning Mar. 12.
Central Generating System for University of Michigan.* (27) Mar. 13.
The Double-Audion Type of Receiver.* A. H. Taylor. (27) Mar. 13.
Factors in Rate-Making. Arthur S. Ives. (27) Serial beginning Mar. 13.
The Relation of the Horse-Power to the Kilowatt. (From *Circular 34*, U. S. Bureau of Standards.) (19) Mar. 13.
Polyphase Commutator Machines and Their Application. N. Shuttleworth. (77) Mar. 15.
Conditions Affecting the Variations in Strength of Wireless Signals.* E. W. Marchant. (77) Mar. 15.
Laying a 6-Ft. Pipe Tunnel Across the Milwaukee River.* (13) Mar. 18.
On the Measurement of Alternating Electric Currents of High Frequency. Albert Campbell and D. W. Dye. (Abstract of paper read before the Royal Soc.) (73) Mar. 19.

* Illustrated.

Electrical—(Continued).

Hydroelectric Development at Cohoes, N. Y.* (27) Mar. 20.
Radiotelegraphy Without Elevated Antennas.* Charles A. Culver and John A. Riner. (27) Mar. 20.
Photometry of Incandescent Lamps.* J. W. Roper. (Paper read before the Yorkshire, England, Junior Gas Assoc.) (24) Mar. 22.
Auxiliary Station for Transmission System.* Curtis A. Mees. (27) Mar. 27.
The Principles of Radio-Telephony.* John L. Hogan, Jr. (46) Mar. 27.
Class Rates for Light and Power Systems or Territories.* Frank G. Baum. (42) Apr.
Continuous Waves in Long Distance Radio Telegraphy.* L. F. Fuller. (42) Apr.
Electrical Precipitation. F. G. Cottrell. (42) Apr.
The Flow of Energy.* Robert A. Philip. (42) Apr.
Mill Controllers.* H. F. Stratton. (42) Apr.
Practical Applications of Electrical Precipitation and Progress of the Research Corporation. Linn Bradley. (42) Apr.
Regulation of Electrotyping Solutions. (From *Circular No. 52,* U. S. Bureau of Standards.) (105) Apr.
The Theoretical and Experimental Consideration of Electrical Precipitation. A. F. Nesbit. (42) Apr.
A New Printing Telegraph System.* Paul M. Rainey. (27) Apr. 3.
Der Schnellregler und der Eilregler der Siemens-Schuckertwerke.* G. Ernst Grau. (41) Feb. 11.
Wechselstrompufferung.* L. Schröder. (41) Serial beginning Feb. 11.
Die Starkstrom-Elektrotechnik in Gruppe 33B an der Schweiz. Landesausstellung. Bern, 1914.* W. Kummer. (107) Feb. 13.
Schutz von Schwachstromleitungen gegen Starkstrom.* Fritz Schröter. (41) Feb. 18.
Eine gefahrlose metallische Röntgenröhre.* L. Zehnder. (107) Feb. 20.
Die elektrische Durchschlagsfestigkeit von flüssigen, halbfesten und festen Isolierstoffen in Abhängigkeit vom Druck.* F. Kock. (41) Serial beginning Feb. 25.
Verfahren zur Erlangung sinusförmiger Spannungskurven bei ein- und mehrphasigen Wechselstromdynamos.* W. Seeman. (41) Mar. 4.
Dielektrische Eigenschaften von verschiedenen Isolierstoffen.* Karl Willy Wagner. (41) Serial beginning Mar. 11.

Marine.

Small Screw Propellers.* D. H. Jackson. (Paper read before the Inst. of Marine Engrs.) (47) Feb. 26.
The Modern Submarine in Naval Warfare.* R. H. M. Robinson. (3) Mar.
The Governing of Marine Steam Turbines.* (12) Mar. 5.
Failure of British Steel Ship Plates.* W. J. B. Wilson. (Paper read before the Northeast Coast Institution of Engrs. and Shipbuilders.) (20) Mar. 18.
Robinson Marine Superheater.* (12) Mar. 19.
The Modern Submarine.* C. A. Ward. (From *Journal* of the Am. Soc. of Marine Draftsmen.) (9) Apr.
The Submarine at Sea.* (46) Apr. 3.
Le Transport des Mines Marines par les Courants sous l'Action de la Houle. E. Bertin. (33) Feb. 27.
Spannungsmessungen an Bord von Schiffen.* Siemann. (48) July 18.

Mechanical.

The Metaline Plant of the Inland Portland Cement Co., Metaline Falls, Wash.* Milo W. Krejci. (56) Vol. 46.
The Great Falls Flue System and Chimney.* C. W. Goodale and J. H. Klepinger. (56) Vol. 46.
Cement Materials and the Manufacture of Portland Cement in Montana. W. H. Andrews. (56) Vol. 46.
The Use of Pulverized Coal as a Fuel for Metallurgical Furnaces. H. R. Barnhurst. (56) Vol. 47.
The Cleaning of Blast-Furnace Gas.* W. A. Forbes. (56) Vol. 47.
The Scoria Process for the Manufacture of Fine-Ore Briquettes, Flue-Dust Briquettes, and Slag Brick for Building Purposes. Ernest Stütz. (56) Vol. 47.
Note on the Utilization of the Waste Heat of Regenerative Furnaces. George C. Stone. (56) Vol. 47.
The Generation of Steam by Waste Heat from Furnaces.* F. Peter. (Translated by R. W. Raymond.) (56) Vol. 47.
The Slagging Gas Producer. William Hutton Blauvelt. (56) Vol. 47.
The Briquetting of Flue-Dust in the United States by the Schumacher Process.* Felix A. Vogel and A. M. Tweedy. (56) Vol. 47.
Air Conditioning.* J. Irvine Lyle. (2) Jan.
Bituminous Coals; Predetermination of Their Clinkering Action by Laboratory Tests.* F. C. Hubley. (2) Jan.

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* Illustrated.

Mechanical—(Continued).

The Burmeister and Wain Oil Engine.* (11) Feb. 19.
The High-Speed Gas Engine.* R. Embleton. (Paper read before the Junior Institution of Engrs.) (47) Feb. 19.
Malleable Iron for Automobiles. Richard Moldenke. (Paper read before the Am. Soc. of Automobile Engrs.) (47) Feb. 19.
Swiss Turbo-Generator Sets.* (12) Feb. 19.
The Useful Recovery of Heat Losses in Internal-Combustion Engines.* J. B. Meriam. (Paper read before the Cleveland Eng. Soc.) (47) Feb. 19.
The Dumping of Refuse by Aerial Ropeway.* A. Varty. (Paper read before the North of England Branch of the National Assoc. of Colliery Mgrs.) (22) Feb. 26.
The Floating Crane Failure at the Panama Canal.* (11) Feb. 26.
Telpher Coal-Handling Plant.* (11) Serial beginning Feb. 26.
Utilisation of Blast-Furnace Slag.* (22) Feb. 26.
The Cost of Bus Operation.* (17) Feb. 27.
Industrial Uses of Fuel Oil.* F. B. Dunn. (111) Serial beginning Feb. 27.
Application of Electricity to the Ore-Handling Industry.* C. D. Gilpin. (42) Mar.
Electric Drive for Economic Operation and Development of Cement Mills.* J. Benton Porter. (55) Mar.
Factors in Hardening Tool Steel.* John A. Mathews and Howard J. Stagg, Jr. (55) Mar.
Gas Producers. R. H. Fernald. (Abstract of paper read before the Cleveland Eng. Soc.) (45) Mar.
The Jitney Bus.* Frank Reed. (60) Mar.
Line Disturbance Caused by Special Squirrel-Cage and Wound-Rotor Motors when Starting Elevators and Hoists. J. C. Lincoln. (42) Mar.
A New Building Material (for Manufacturing Brick).* (60) Mar.
A New Volume Regulator for Air Compressors.* Ragnar Wikander. (55) Mar.
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PAPERS AND DISCUSSIONS

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TEMPERATURE CHANGES IN MASS CONCRETE

By Charles H. Paul, M. Am. Soc. C. E.,
and A. B. Mayhew, Assoc. M. Am. Soc. C. E.

To be Presented May 5th, 1915.

Synopsis.

If free to move, all structural materials undergo changes in length due to changes in temperature, and, if not free to move, a stress in the material is produced equal to that required to produce the deformation in length. In order to determine the extent of this change in length, or the corresponding stress in the material, it becomes necessary to know the range in temperature to which the material is to be subjected. In large masses of masonry, the changes in temperature at varying distances from the exposed faces are not known, and, largely due to this fact, the changes in length of large masses of masonry, together with the actual internal distribution of temperature stresses, are among the most indefinite factors in the design of such structures.

The purpose of this paper is to place before the Society the results, thus far obtained, of experiments being made to determine the changes of temperature in the concrete of the Arrowrock Dam, together with a description of the apparatus used.

Although some of these experiments have been in progress for more than a year, the greater part of the results is still affected by the high

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

temperatures produced by chemical action while the cement is setting, and, therefore, this paper will be largely descriptive, and should be considered in the light of a preliminary paper.

Among the conclusions already reached are the following:

(1) Large bodies of concrete deposited rapidly during a summer season develop a temperature of from 90 to 95° within a period of about 30 days, and maintain nearly that temperature for several months.

(2) In the case of concrete 1 ft. from an exposed face, there is a daily variation in temperature of about 2° when the daily variation in the temperature of the air is about 50 degrees.

(3) In the case of concrete 2 ft. from an exposed face, there is a daily variation of less than 1° when the daily variation in the temperature of the air is about 50 degrees.

(4) In the case of concrete 3.5 ft. from an exposed face, no daily variation in temperature is apparent when the daily variation in the temperature of the air is about 50 degrees.

(5) The seasonal variation in the temperature of concrete 3.5 ft. from an exposed face is about 32° when the seasonal variation in the mean daily temperature of the air is about 75 degrees.

(6) The experiments have not yet been carried far enough to show the seasonal variation at other distances from exposed faces, but it is probable that they become very much less as the distance from the face increases.

Introductory.

The Arrowrock Dam, being constructed by the United States Reclamation Service near Boise, Idaho, and on which the writers are employed as Construction Engineer and Principal Engineering Assistant, respectively, is of the gravity type, and is curved in plan to a radius of about 662 ft. at the center of the crest. Inasmuch as little information is available concerning temperature changes in large bodies of masonry, and as this is often an important factor in the design of high dams, it was decided, for the benefit of future designs and to further engineering knowledge in general, to install in the Arrowrock Dam devices for recording the temperature of the masonry at various distances from the exposed faces. Nine of these devices

have been installed to date, and have been in place for periods ranging from about 3 to about 19 months.

It will probably be several years before a complete study can be made of the temperature changes, but, as the data on this subject are so meager, it is deemed advisable at this time to place before the Society the results thus far obtained, together with a description of the apparatus used and the principal conditions affecting the temperature readings.

Temperature Recording Device.

The electrical resistance thermometer was considered to be the simplest and best apparatus for the purpose contemplated. The fundamental principle of the resistance thermometer is the fact that a change in the temperature of a metal causes a corresponding change in its electrical resistance.

The thermometer, or resistance bulb, consists of a coil of fine copper wire, encased in a metal tube about $\frac{3}{8}$ in. in diameter and about 5 in. long. One end of the tube is closed and the other end is soldered, with a water-proof joint, to a lead-covered cable containing the wires connecting the bulb with the switch-board to which the temperature indicator is attached.

The leads connecting the thermometer to the switch-board are constructed as follows: Each wire of the cable is insulated with three wrappings of silk, thoroughly saturated with beeswax after wrapping. The three strands of which the cable is formed are then brought together and encased in one wrapping of cotton and one of braid similarly water-proofed. The cable is then drawn into the lead armor, the latter being soldered directly to the brass tube containing the resistance coils.

The temperature indicator consists of a modified Wheatstone bridge calibrated to read temperature units instead of ohms. The dial is direct reading in degrees, Fahrenheit, with a range from zero to 100°, and, by interpolation, can be read to about 0.1 degree. It is claimed by the manufacturers that accurate results may be obtained with the indicator to 0.25°, and with the apparatus as a whole to 0.5 degree. From comparative readings made with an ordinary mercury thermometer, it appears that these claims are fully justified. The apparatus installed at Arrowrock was obtained from the Leeds and Northrup Company, of Philadelphia.

General Method of Placing Thermometers.

The resistance bulbs were placed in one vertical plane, but at various elevations and at different distances from the exposed faces of the dam. Ultimately, there will be one resistance bulb near the top of the dam, one about half way between the original river level and the bed-rock foundation, and eight more at intermediate elevations. When each resistance bulb was placed in the masonry, the lead-covered cable was encased in $\frac{1}{2}$-in. steel tubing and carried to an inspection gallery in the body of the dam, where the switch-board and indicator were installed. When the concrete in the dam was built to the approximate elevation at which it was desired to place a resistance bulb, concreting in that immediate vicinity was usually discontinued temporarily, and the concrete was allowed to set for a few hours. The bulb and conduit were then embedded in a trench, about 6 in. deep, excavated in the concrete, great care being taken to provide a soft, uniform bed for the bulb, and to cover it carefully. Deviation from this practice is mentioned, and further details are given in the following paragraphs.

General Conditions Affecting the Temperature Readings.

The greater part of the masonry in the body of the dam is composed of sand-cement, river sand, river gravel, and river cobbles, mixed in the proportions of about $1:2\frac{1}{2}:5:2\frac{3}{4}$, although these proportions have been varied from time to time. Unless otherwise specified, it is to be understood that the cement used in all concrete is sand-cement. In all cases, the sand-cement is the product of grinding, in a tube mill, Portland cement and pulverized granite, mixed in the proportions of 45% of granite, and 55% of Portland cement, by weight. The faces of the dam are composed of a richer concrete than that used in the body; the thickness of the face mix varying with the elevation above bed-rock. In most cases, the cement used in the face mix is composed of about 76% of sand-cement and about 24% of straight Portland cement by volume, and the concrete mix is about $1:2:4:2\frac{1}{2}$.

In some cases, concreting was not carried on continuously directly above the thermometers. An attempt has been made to show this condition on the accompanying diagrams, by giving the depth of concrete directly over the thermometers at frequent intervals of time.

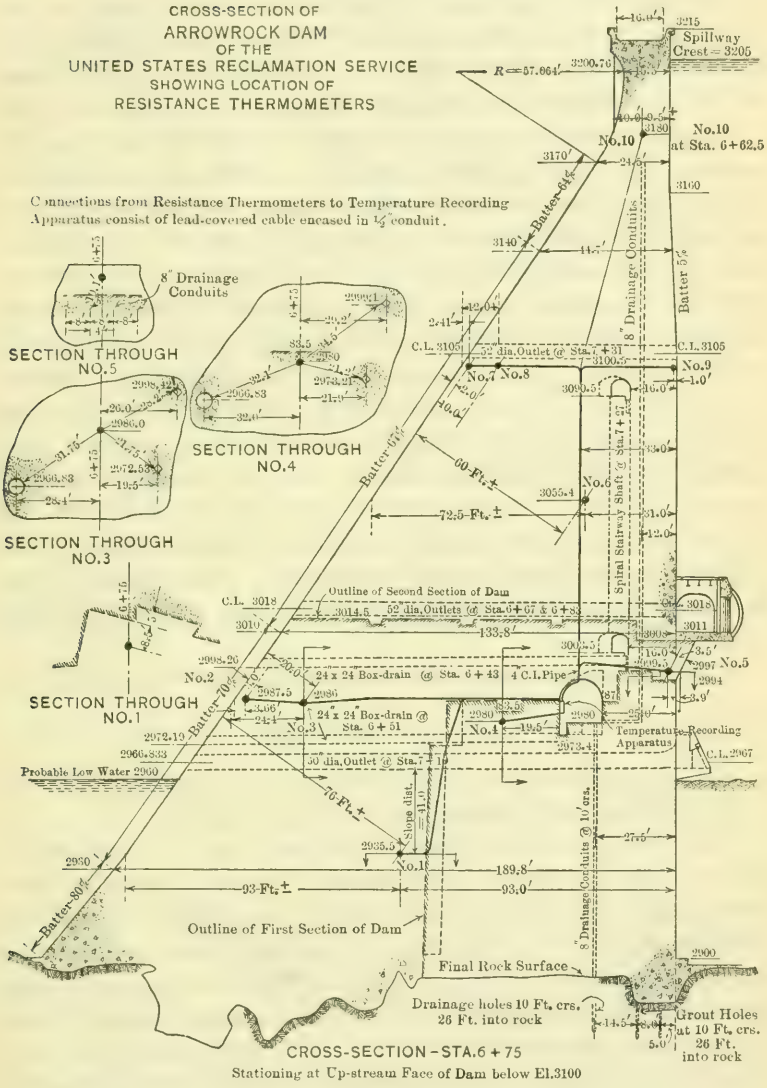

CROSS-SECTION OF
ARROWROCK DAM
OF THE
UNITED STATES RECLAMATION SERVICE
SHOWING LOCATION OF
RESISTANCE THERMOMETERS
Connections from Resistance Thermometers to Temperature Recording Apparatus consist of lead-covered cable encased in ½" conduit.
SECTION THROUGH NO.5
SECTION THROUGH NO.4
SECTION THROUGH NO.3
SECTION THROUGH NO.1
8" Drainage Conduits
Spillway Crest = 3205
No.10 at Sta. 6+62.5
No.7
No.8
No.9
No.6
No.5
No.4
No.3
No.2
No.1
Batter-64"
Batter-67"
Batter-70"
Batter-80"
60-Ft.±
72.5-Ft.±
76-Ft.±
93-Ft.±
Spiral Stairway Shaft @ Sta.7+37
Temperature Recording Apparatus
Outline of Second Section of Dam
Outline of First Section of Dam
Final Rock Surface
Drainage holes 10 Ft. crs. 26 Ft. into rock
Grout Holes at 10 Ft. crs. 26 Ft. into rock
Probable Low Water 2960
CROSS-SECTION - STA.6 + 75
Stationing at Up-stream Face of Dam below El.3100

FIG. 1.

The up-stream face of the dam is an east exposure, and the down-stream face is therefore a west exposure.

Because of the great depth of the dam foundation below the river bed—about 90 ft.—and the correspondingly great danger from floods, it was decided to uncover at first only a portion of the bed-rock along the line of the up-stream face of the dam and to build thereon a wall of concrete having a cross-section heavy enough to form a gravity section at the height to which it was built. After this wall had been brought up to about 40 ft. above the old river bed, the remainder of the bed-rock foundation was uncovered, and concrete was placed to form the full section of the dam. The wall constructed first is known as the "first section of the dam", and is shown in outline on the cross-section, Fig. 1. The concrete in this wall was placed intermittently between November 10th, 1912, and April 15th, 1913. The remainder of the masonry in the dam, to Elevation 3 014.5, as shown in outline on Fig. 1, is known as the "second section of the dam", and was placed between July 10th, and December 1st, 1913. At the end of the 1914 season the top of the masonry will be at about Elevation 3 160, and the concrete placed above the second section and below Elevation 3 160 is known as the "third section of the dam". The masonry in the third section was placed between February 26th, and about December 1st, 1914.

No water had been stored in the reservoir up to the time of this writing.

Explanation of Diagrams.

Fig. 1 is a cross-section of the Arrowrock Dam, showing the location of the resistance thermometers, and is self-explanatory.

Figs. 2 to 7, inclusive, show, to a large scale, the temperature changes in the concrete at Thermometers Nos. 1 to 9, inclusive, for a period of 180 hours after each was placed in the masonry. During this period readings were usually taken at intervals of from 1 to 24 hours, and the temperature of the air shown on these diagrams was platted from the maximum, minimum, and 6.00 P. M. temperatures—or from the maximum, minimum, and 5.00 P. M. temperatures, as the case might be—recorded each day at a United States Weather Bureau station, about ¼ mile from the dam site.

The upper part of Plate VIII shows, to a large scale, the temperature changes in the concrete at Thermometers Nos. 7 and 9 for a

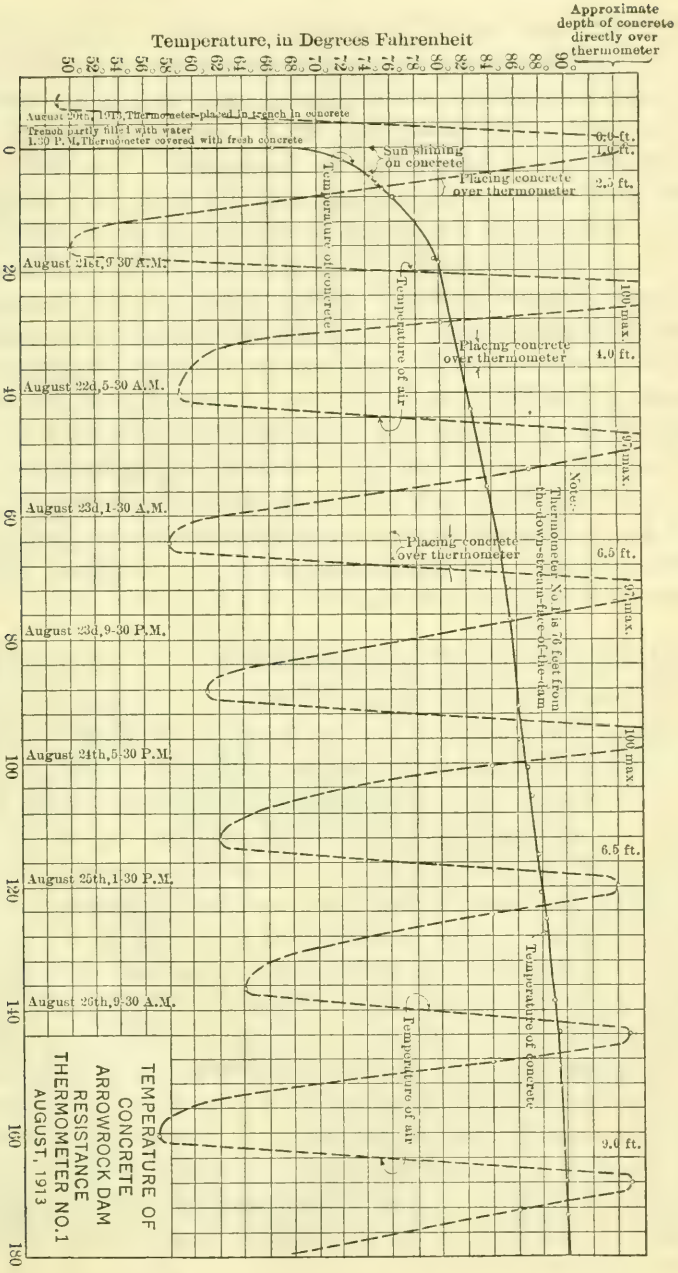

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometer
Time, in Hours
August 20th, 1913. Thermometer placed ½ inch in concrete
French partly filled with water
1:30 P.M. Thermometer covered with fresh concrete
Sun shining on concrete
Placing concrete over thermometer
Temperature of concrete
Temperature of air
August 21st, 9:30 A.M.
August 22d, 5:30 A.M.
August 23d, 1:30 A.M.
August 23d, 9:30 P.M.
August 24th, 5:30 P.M.
August 25th, 1:30 P.M.
August 26th, 9:30 A.M.
0.0 ft.
1.0 ft.
2.5 ft.
4.0 ft.
5.5 ft.
6.5 ft.
9.0 ft.
100 max.
Note.—Thermometer No. 1 is 70 feet from the down-stream face of the dam.
TEMPERATURE OF CONCRETE
ARROWROCK DAM
RESISTANCE THERMOMETER NO. 1
AUGUST, 1913

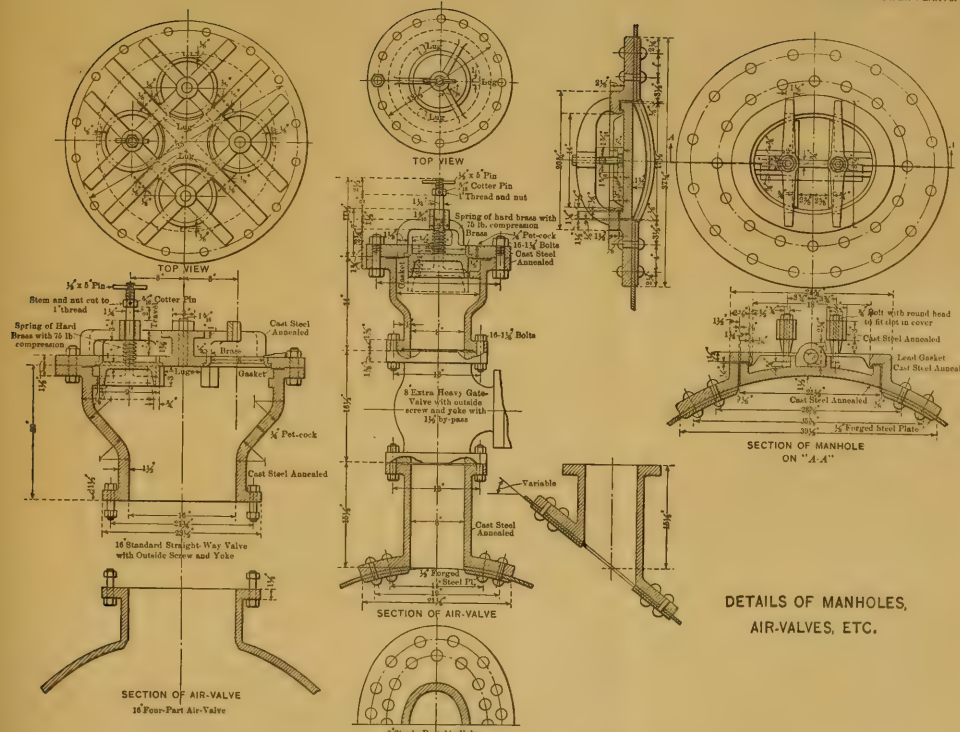
Fig. 2.

period of 48 hours, and at Thermometer No. 5 for a period of 24 hours. At the time these results were recorded, the effect of the chemical action produced by the setting of the concrete had largely disappeared, and the controlling factor seemed to be the temperature of the air outside, as may be seen by reference to the lower part of Plate VIII. The object of the series of readings shown on Fig. 8 was to determine the daily variation in the temperature of the concrete near an exposed face, at a time when the variation in temperature of the air was near a maximum. The temperature of the air, as shown on this diagram, was taken by mercury thermometers, placed on the faces of the dam near the location of each of the resistance thermometers in question, and exposed to the direct rays of the sun. In studying this diagram, it should be borne in mind that Thermometers Nos. 5 and 9 are near the east face of the dam and No. 7 is near the sloping west face.

Plate VIII shows, to a small scale, all results obtained to date. The greater part of the concrete temperatures were platted from weekly readings. The temperature of the air, as platted on these diagrams, is the mean daily temperature.

Detailed Data Concerning Thermometers Nos. 1 to 9, Inclusive.

Thermometer No. 1.—At about 12.00 M. on August 20th, 1913, Resistance Thermometer No. 1 was placed in a trench, about 6 in. deep, excavated in a 1 : 2½ : 6 : 2 mixture of concrete placed about 3.00 P. M. on August 19th. This concrete was originally of the plastic consistency ordinarily used, but, at the time the thermometer was placed, a shallow pool of water had accumulated at the point where it was desired to embed the bulb, and the trench therefore filled with water as fast as it was excavated. At this time the concrete was shaded from the direct rays of the sun by the first section of the dam, and by a lava bluff to the south. While the bulb was lying in the trench and covered with water, the indicator registered a temperature of 66.5° Fahr. At about 1.30 P. M., about 8 cu. yd. of fresh concrete were deposited over and around the bulb to protect it. At this time the sun was shining brightly on the concrete in the vicinity of the bulb, and continued thus until 6.15 P. M. A uniform layer of concrete about 2.5 ft. thick was placed in the immediate vicinity of the bulb between 7.00 and 8.00 P. M. on August 20th. Further details are shown on Fig. 2 and Plate VIII.



DETAILS OF MANHOLES, AIR-VALVES, ETC.

The highest temperature recorded by Thermometer No. 1 was 94.0° Fahr., which reading remained constant from September 20th to 25th, inclusive, at which time there was a mass of 15.5 ft. of concrete directly above the thermometer. Assuming the initial temperature of the concrete as 66.5°, the maximum rise in temperature, due primarily to chemical action, was therefore 27.5°, and occurred during a period of 31 days.

Thermometer No. 1 is 76 ft. from the down-stream face of the dam, about 60 ft. above bed-rock, and about 43 ft. from one of the sluicing outlets. The temperature in this outlet was above freezing throughout the winter of 1913-14.

Thermometers Nos. 2 and 3.—At about 3.00 P. M. on October 19th, 1913, Thermometers Nos. 2 and 3 were placed in a trench, about 6 in. deep, excavated in concrete placed about 1.00 A. M. on the same day. Thermometer No. 2 was placed in a 1:2:4¾:1½ mixture of concrete, of plastic consistency. The cement consisted of approximately 78% of sand-cement and 22% of straight Portland cement. Thermometer No. 3 was placed in a 1:2½:6:2 mixture of concrete, although it is probable that this concrete may have contained a small quantity of the face mixture in which No. 2 was embedded. In water content, this concrete, near the surface, at least, was above normal.

Although the greater part of the masonry in the dam was exposed to the direct rays of the sun on October 19th, the portion in which the thermometers were placed was, at that time and during all the winter, in the shadow of the mountains to the south of the dam.

At 3.00 P. M. on October 19th, while the resistance bulbs were lying in the trenches, but before being covered with concrete, the registration of No. 2 was 57°, and that of No. 3 was 52.8 degrees. The bulbs were then covered with concrete to a depth of about 6 in., to protect them from injury. At about 9.00 A. M. on October 20th, about 2 ft. of the regular 1:2½:6:2 mixture of concrete was placed over Bulb No. 3, and at about 10.00 A. M. No. 3 registered 49.5° and No. 2 55 degrees. Between 10.00 P. M. on October 20th and 2.00 A. M. on October 21st, about 4 ft. of the face mix concrete described above was placed over Bulb No. 2, and the regular mixture of concrete was brought up to a total height of about 4 ft. above Bulb No. 3. Further details are shown on Fig. 3 and Plate VIII.

About October 25th the temperature readings on Thermometer No. 2 began to show irregularities, and it soon became apparent that there was a partial short circuit. Readings could be taken with a weak battery current, but were undoubtedly not entirely accurate. It seems probable that moisture penetrated the joint between the bulb and the lead cable. It was thought that the continual application of current while the insulation was in a moist condition might permanently injure the insulation; and, as the extent of the inaccuracy in the readings could not be determined, it was decided to discontinue the readings temporarily in the hope that the moisture would ultimately be absorbed by the concrete and the apparatus return to its normal condition. Although some improvement has been shown in the trial readings made from time to time, the results obtained from this thermometer are still unreliable.

The highest temperature recorded by Thermometer No. 2 was 74.5° on October 25th. Assuming the initial temperature of the concrete as 55°, the maximum rise in temperature was therefore 19.5°, and occurred in a period of 5 days. As this thermometer is only 3 ft. from an exposed face, it is probable that the air temperature, which was low at this time, counteracted more or less the heat generated by chemical action.

The highest temperature recorded by Thermometer No. 3 was 86.2°, which reading remained constant from December 4th to 22d, inclusive. Assuming the initial temperature of the concrete as 49.5°, the maximum rise in temperature, due principally to chemical action, was therefore 36.7°, and is the greatest rise recorded by any of the thermometers under consideration. This rise occurred during a period of 45 days. At this time there was a depth of 28.5 ft. of concrete directly over the thermometer.

Thermometers Nos. 2 and 3 are 3.0 ft. and 20.0 ft., respectively, from the down-stream face of the dam.

Thermometer No. 4.—About 5.00 P. M. on March 20th, 1913, Thermometer No. 4, the first one installed in the dam, was embedded about 18 in. in a 2.5-ft. layer of concrete placed during the previous hour or so. After the bulb was placed, concreting in this vicinity was continued until the surface was brought up to about 3.5 ft. above the bulb, at which elevation the surface of the concrete, directly above the bulb, remained for the following 7 months. As will be seen by

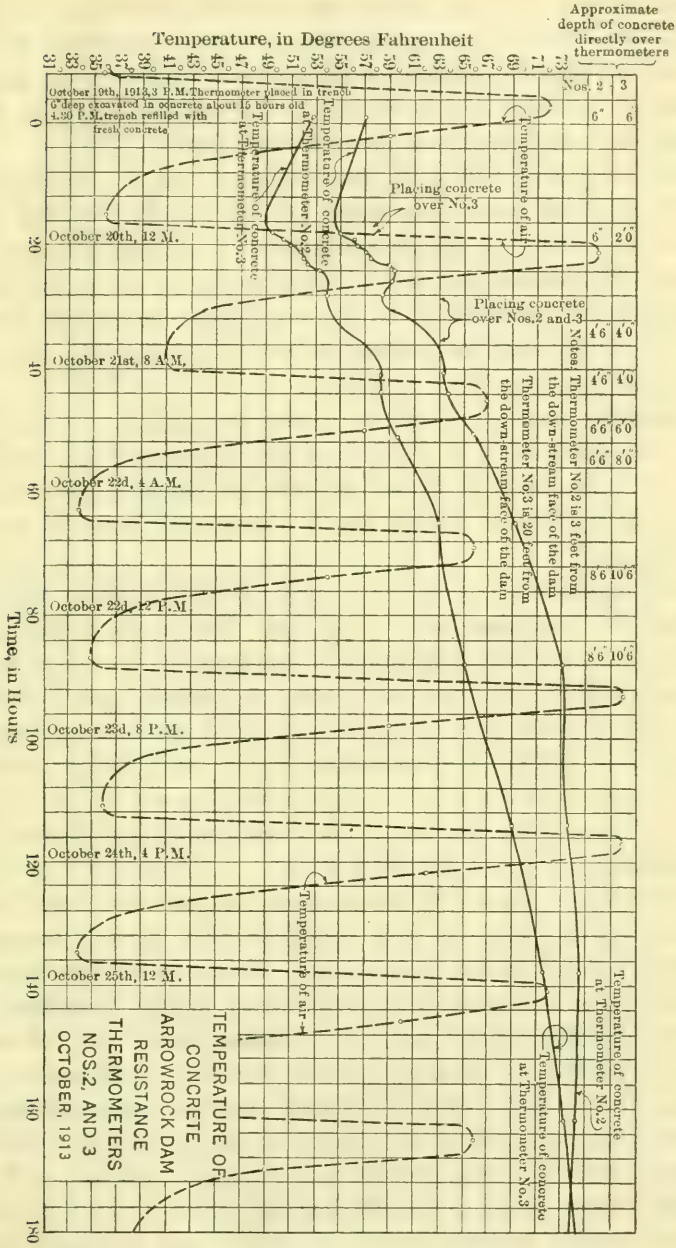

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometers
Nos. 2 3
6" 6"
2'0"
4'6" 4'0"
6'6" 6'0"
8'6" 10'6"
October 19th, 1913, 3 P.M. Thermometer placed in trench 6' deep excavated in concrete about 15 hours old 4.30 P.M. trench refilled with fresh concrete
October 20th, 12 M.
October 21st, 8 A.M.
October 22d, 4 A.M.
October 22d, 12 P.M.
October 23d, 8 P.M.
October 24th, 4 P.M.
October 25th, 12 M.
Time, in Hours
Temperature of air
Temperature of concrete at Thermometer No.2
Temperature of concrete at Thermometer No.3
Placing concrete over No.3
Placing concrete over Nos.2 and 3
Notes. Thermometer No.2 is 3 feet from the down-stream face of the dam
Thermometer No.3 is 20 feet from the down-stream face of the dam
TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETERS NOS.2, AND 3 OCTOBER, 1913

Fig. 3.

reference to Fig. 1, a key in the surface of the concrete in the first section of the dam was constructed directly over Thermometer No. 4. The concrete forming the sides of this key was brought up to a height of about 7 ft. above No. 4 on March 22d, and concreting in that vicinity was then discontinued for several months.

The initial temperature of the concrete in which Thermometer No. 4 was placed was not recorded, but, by reference to Fig. 4 it will be seen that the initial temperature was probably about 44 degrees. The highest temperature recorded during the period when chemical action appeared to be the controlling temperature factor was on April 4th, when the temperature reached 64.6 degrees. This rise occurred during a period of 15 days.

Concreting immediately above Thermometer No. 4 was resumed on October 20th, 1913, at which time the thermometer had been in place for 7 months, and, as will be seen by reference to the upper half of Plate VIII, a decided rise in temperature began to appear 6 days later.

Thermometer No. 4 is 19.5 ft. from the inspection gallery and 58.5 ft. from the up-stream face of the dam. During the period from March 21st to October 20th, 1913, the thermometer was 3.5 ft. below the surface of the concrete, and during the period from November 15th, 1913, to April, 1914, it was 34.5 ft. below the surface. During the winter of 1913-14, the temperature in the inspection gallery was slightly warmer than that of the air outside, but was below freezing during a considerable portion of the time. Additional details may be obtained from Fig. 4 and Plate VIII.

Thermometer No. 5.—At 9.30 A. M. on October 29th, 1913, Thermometer No. 5 was placed in a trench, about 6 in. deep, excavated in concrete placed during the evening of October 28th. This concrete was of plastic consistency, and was mixed in the proportions of $1:2\frac{3}{4}:4:1\frac{1}{2}$. The cement consisted of approximately 78% of sand-cement and 22% of straight Portland cement. Considerable reinforcing steel for the trash-rack structure was also embedded in this concrete, and extended out to within 3 in. of the up-stream face of the dam. After covering the bulb with concrete to a depth of 6 in., the temperature recorded was 43 degrees. About 1 ft. of the face mix of concrete described above was placed over the thermometer

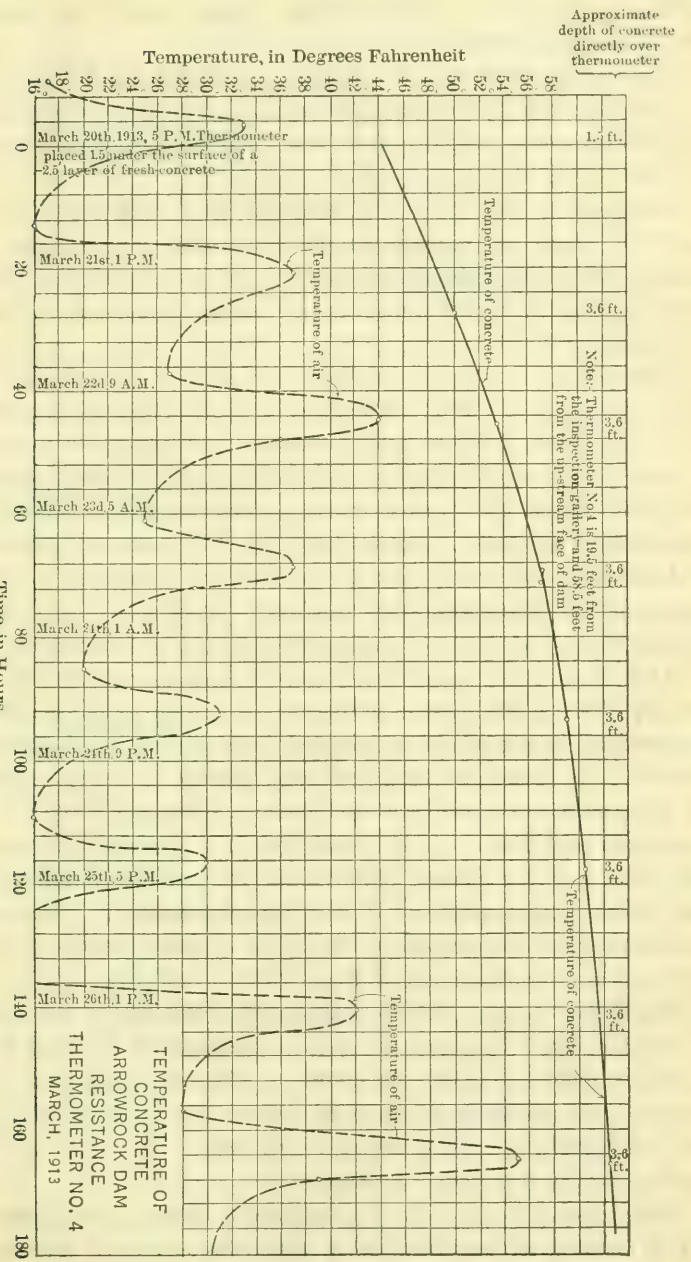

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometer
March 20th 1913, 5 P.M. Thermometer placed 1.5 under the surface of a 2.5 layer of fresh concrete
March 21st 1 P.M.
March 22d 9 A.M.
March 23d 5 A.M.
March 24th 1 A.M.
March 24th 9 P.M.
March 25th 5 P.M.
March 26th 1 P.M.
Temperature of air
Temperature of concrete
Temperature of concrete
Temperature of air
Note.—Thermometer No. 4 is 19.5 feet from the inspection gallery and 58.5 feet from the upstream face of dam
1.5 ft.
3.6 ft.
3.6 ft.
3.6 ft.
3.6 ft.
3.6 ft.
3.6 ft.
TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETER NO. 4 MARCH, 1913
Time, in Hours

Fig. 4.

during the evening of October 30th. Further details are shown on Fig. 5 and Plate VIII.

The highest temperature recorded was 69.8°, which was reached on November 3d. The maximum rise in temperature, due primarily to chemical action, was therefore 26.9°, and occurred during a period of 5 days.

Thermometer No. 5 is 3.5 ft. from the up-stream face of the dam, and 3 ft. above the concrete in the first section of the dam. It is probable that the temperature of the air, which was low, had an effect on this thermometer similar to that mentioned in the case of Thermometer No. 2.

Thermometer No. 6.—At 12.00 M. on June 23d, 1914, Thermometer No. 6 was set in the usual manner, in concrete placed about 2.00 A. M. on the same day. Between 1.00 and 2.30 P. M. on the 25th, the surface of the concrete was brought up to a point about 3 ft. above the thermometer. About this time it was discovered that there was an open circuit in the apparatus. A hole, about 6 ft. in diameter at the top, 2 ft. in diameter at the bottom, and 3.5 ft. deep, was excavated in the concrete, and the thermometer was removed. The trouble was located in the resistance coil inside of the metal tube, and was remedied.

At 9.00 A. M. on June 26th, a bed of concrete, about 6 in. deep, was deposited in the bottom of the hole excavated in the concrete on the previous afternoon, and on this bed Thermometer No. 6 was placed. At 9.30 A. M., after the bulb had been covered to a depth of 6 in., the temperature recorded was 61.5 degrees. Between 11.00 P. M. on June 26th and 2.30 A. M. on June 27th, the remainder of the hole was filled, and the general elevation of the surface of the concrete was brought up to a point 3.5 ft. above the thermometer. The concrete was mixed in the proportions $1:2\frac{1}{2}:5\frac{1}{2}:2\frac{3}{4}$. Further details are shown on Fig. 6 and the lower half of Plate VIII.

The highest temperature recorded by No. 6 was 96.2°, and occurred on October 5th. This was the highest temperature recorded by any of the thermometers under consideration. The maximum rise in temperature, due primarily to chemical action, was therefore 35.7°, and this rise occurred during a period of 101 days. (A rise of 32.2° was recorded during a period of 31 days.)

Thermometer No. 6 is 31 ft. from the up-stream face of the dam, and on October 1st was about 80 ft. below the surface of the concrete.

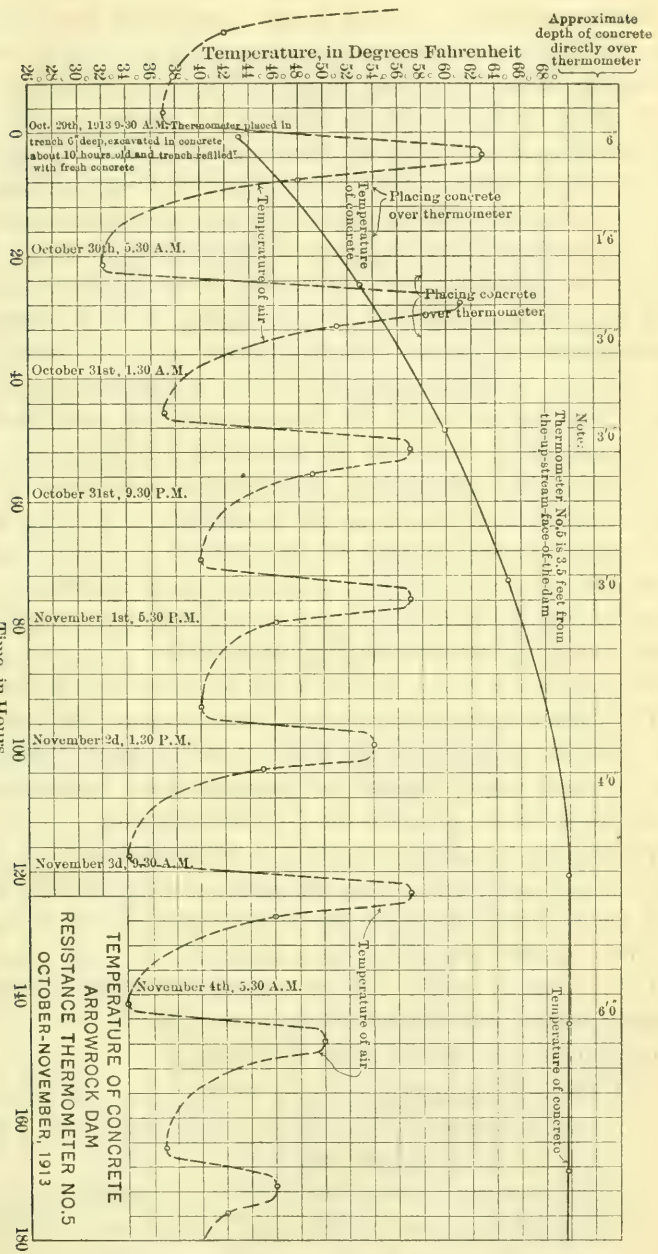

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometer
Time, in Hours
Oct. 29th, 1913 9:30 A.M. Thermometer placed in trench 6' deep, excavated in concrete about 10 hours old and trench refilled with fresh concrete
October 30th, 5.30 A.M.
October 31st, 1.30 A.M.
October 31st, 9.30 P.M.
November 1st, 5.30 P.M.
November 2d, 1.30 P.M.
November 3d, 9.30 A.M.
November 4th, 5.30 A.M.
Temperature of air
Temperature of concrete
Placing concrete over thermometer
Placing concrete over thermometer
Temperature of air
Temperature of concrete
Note: Thermometer No. 5 is 2.5 feet from the up-stream face of the dam
TEMPERATURE OF CONCRETE
ARROWROCK DAM
RESISTANCE THERMOMETER NO. 5
OCTOBER-NOVEMBER, 1913

Fig. 5.

Thermometers Nos. 7, 8, and 9.—On August 20th, 1914, Thermometers Nos. 7, 8, and 9 were placed in a trench, about 2½ ft. deep and 3 ft. wide, formed in concrete placed on August 8th and 9th. At 1.30 P. M., while the bulbs were in place in the trench, but not covered with concrete, the readings of Thermometers Nos. 7, 8, and 9 were 77°, 77°, and 73.5°, respectively. A bed of fresh concrete, about 6 in. in depth, was then placed in the bottom of the trench and under the thermometers. Then concrete was deposited over them to a depth of 1.5 ft. in the case of Nos. 7 and 8, and 2.0 ft. in the case of No. 9. At 3.20 P. M., when this had been accomplished, the readings of the thermometers were 74°, 75°, and 75.5°, respectively. No more concrete was placed over these thermometers until September 4th. On September 9th, the surface of the concrete had been brought up to a point about 13 ft. above the thermometers, and on October 1st, to a point about 36 ft. above them. The concrete in which Nos. 7 and 9 were embedded was mixed in the proportions 1:2:4:2½, and the cement used consisted of approximately 76% of sand-cement and 24% of straight Portland cement. The concrete in which No. 8 was embedded was mixed in the proportions of 1:2½:5:2, although it is probable that this concrete contained more or less of the face mixture. Further details are shown on Fig. 7 and the lower half of Plate VIII.

The highest temperature recorded by Thermometer No. 7 was 77.6°, and occurred on August 21st, about 24 hours after it was placed. Another slight rise in temperature is shown about September 8th, or shortly after the concreting above No. 7 was resumed.

The highest temperature recorded by Thermometer No. 9 was 78°, and occurred on August 21st, and again on August 22d, although there was an intermediate reading of 77.8 degrees. This thermometer also shows a slight rise about September 8th.

The unusually small rise in temperature, due primarily to chemical action, recorded by Thermometers Nos. 7 and 9, was undoubtedly caused by the following conditions:

(1) These thermometers are only 2.0 ft. and 15 ft., respectively, from the faces of the dam, and were covered to a depth of only 1.5 ft. and 2.0 ft., respectively.

(2) The concrete forming the sides of the trench in which the thermometers were placed was deposited 11 or 12 days previous to the time they were put in position.

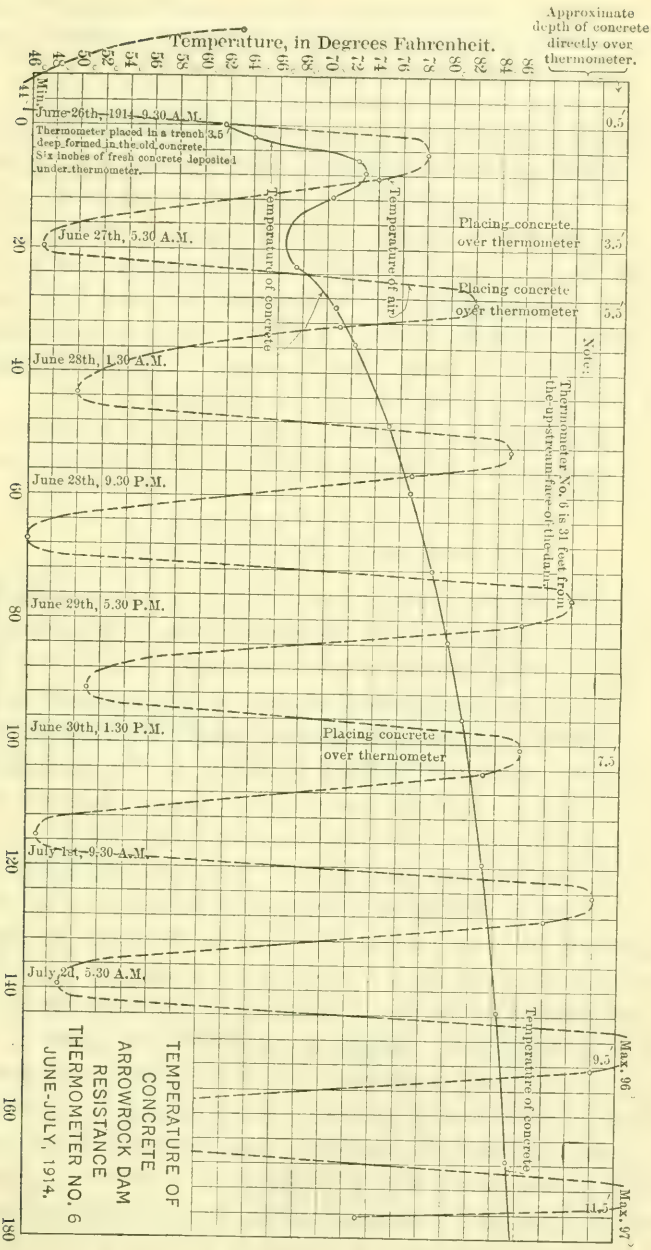

Temperature, in Degrees Fahrenheit.
Approximate depth of concrete directly over thermometer.
Min. 44
Max. 96
Max. 97
June 26th, 1914—9.30 A.M.
Thermometer placed in a trench 3.5 deep formed in the old concrete. Six inches of fresh concrete deposited under thermometer.
June 27th, 5.30 A.M.
June 28th, 1.30 A.M.
June 28th, 9.30 P.M.
June 29th, 5.30 P.M.
June 30th, 1.30 P.M.
July 1st, 9.30 A.M.
July 2d, 5.30 A.M.
Placing concrete over thermometer
Placing concrete over thermometer
Placing concrete over thermometer
Temperature of concrete
Temperature of air
Temperature of concrete
Note: Thermometer No. 6 is 31 feet from the upstream face of the dam.
0.5
3.5
5.5
7.5
9.5
11.5
Time, in Hours.
TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETER NO. 6 JUNE-JULY, 1914.

Fig. 6.

(3) After a small quantity of fresh concrete had been deposited in the trench and over the thermometers, no more was placed in this vicinity for a period of 8 days.

The highest temperature recorded by Thermometer No. 8 was 91.6°, and this reading remained constant from September 21st to 24th, inclusive. The maximum rise, due primarily to chemical action, was therefore 16.6°, and occurred within a period of 32 days. It would probably be more nearly correct to consider the rise due to chemical action as taking place between September 4th and 21st, in which case the rise would be about 21° in 17 days, as shown on the lower half of Plate VIII. On October 1st, there was a depth of approximately 35 ft. of concrete directly over this thermometer.

Thermometers Nos. 7 and 8 are 2.0 and 10.0 ft., respectively, from the down-stream face of the dam, and No. 9 is 1.0 ft. from its up-stream face.

Discussion of Results Obtained.

The effect of the temperature of the air in counteracting the heat generated in the concrete by chemical action is illustrated by the fact that all the comparatively low maximum temperatures recorded by several of the thermometers occurred at points where the distance to an exposed face was comparatively small, and that all the high maximum temperatures recorded by a few of the thermometers occurred at points where the distance to an exposed face was considerable, and over which concrete to a considerable depth was placed during a comparatively short period. Therefore, it appears probable that the speed of placing has fully as much influence on the maximum temperature of mass concrete as either the season of placing or the richness of the concrete.

The duration of the rise in temperature, due primarily to chemical action, is undoubtedly largely dependent on the speed of placing, season of the year, consistency, brand of cement, and quantity of cement per cubic yard of masonry. From incomplete results obtained by placing a mercury thermometer inside the forms of the regulating outlets, it was found that a temperature of 99° was produced in about 24 hours by the chemical action, caused by the setting of the 1:2 Portland cement mortar composing the lining of these outlets.

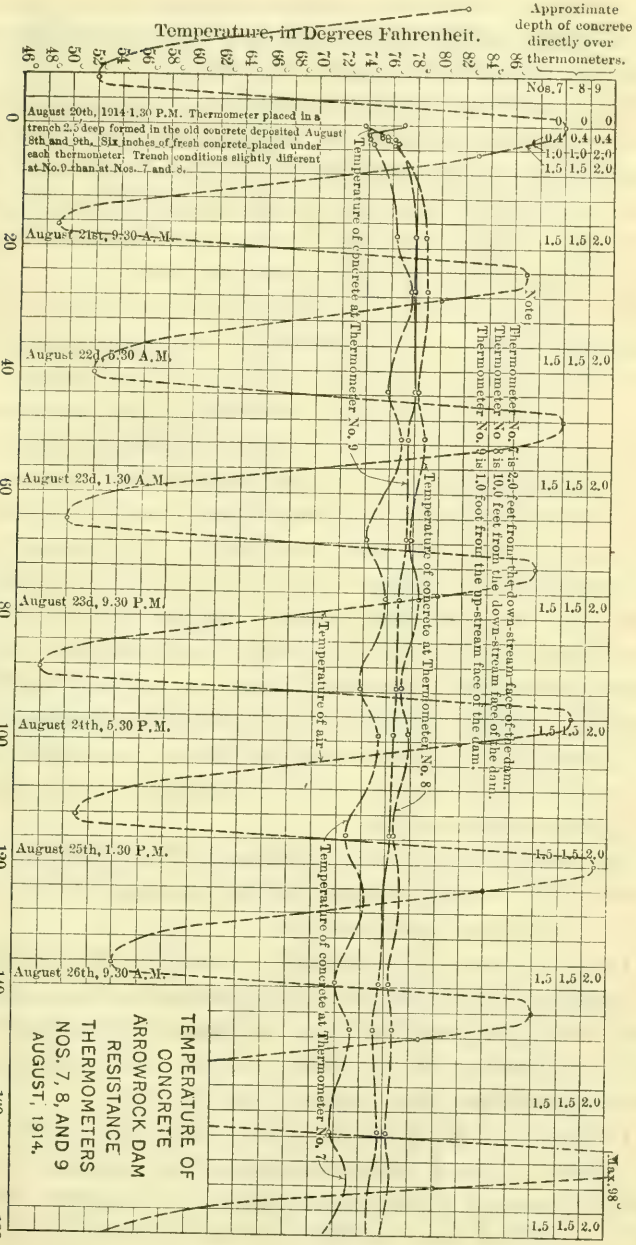

Temperature, in Degrees Fahrenheit.
46° 48° 50° 52° 54° 56° 58° 60° 62° 64° 66° 68° 70° 72° 74° 76° 78° 80° 82° 84° 86°
Approximate depth of concrete directly over thermometers.
Nos. 7 - 8 - 9
0 20 40 60 80 100 120 140 160 180
Time, in Hours.
August 20th, 1914 1.30 P.M. Thermometer placed in a trench 2.5 deep formed in the old concrete deposited August 8th and 9th. Six inches of fresh concrete placed under each thermometer. Trench conditions slightly different at No. 9 than at Nos. 7 and 8.
0 0 0
0.4 0.4 0.4
1.0 1.0 2.0
1.5 1.5 2.0
August 21st, 9.30 A.M. 1.5 1.5 2.0
August 22d, 5.30 A.M. 1.5 1.5 2.0
August 23d, 1.30 A.M. 1.5 1.5 2.0
August 23d, 9.30 P.M. 1.5 1.5 2.0
August 24th, 5.30 P.M. 1.5 1.5 2.0
August 25th, 1.30 P.M. 1.5 1.5 2.0
August 26th, 9.30 A.M. 1.5 1.5 2.0
1.5 1.5 2.0
1.5 1.5 2.0
Max. 98°
Temperature of concrete at Thermometer No. 9
Temperature of concrete at Thermometer No. 8
Temperature of air
Temperature of concrete at Thermometer No. 7
Note.
Thermometer No. 7 is 20 feet from the down-stream face of the dam.
Thermometer No. 8 is 10.0 feet from the down-stream face of the dam.
Thermometer No. 9 is 1.0 foot from the up-stream face of the dam.
TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETERS NOS. 7, 8, AND 9 AUGUST, 1914.

Fig. 7.

It is probable that a higher temperature than that recorded actually existed in the masonry. The consistency of this mortar was somewhat dryer than that ordinarily used in the concrete.

Fig. 8 shows that a daily variation in temperature occurs in the masonry at a distance 2 ft. or less from the faces, but apparently disappears at a depth of 3.5 ft. It will be seen that at a depth of 1 ft. the maximum temperature of the concrete during any 24 hours occurs about 12 hours later than the maximum temperature of the air, and at a depth of 2 ft. it is about 18 hours later.

The results obtained from Thermometer No. 5 serve as the best illustration of the seasonal variation in the temperature of the concrete, and are shown on Plate VIII. These results cover a period of 1 year, and show a seasonal variation of 32.2°, at a distance of $3\frac{1}{2}$ ft. from an exposed face, the lowest temperature being recorded on February 21st, and the highest on August 17th.

The results obtained from Thermometer No. 4 between about April 15th and October 15th, 1913, also illustrate the seasonal variation, and are shown on the upper half of Plate VIII. As these readings were taken at much more frequent intervals than in the case of Thermometer No. 5, they show more conclusively that high or low temperatures of the air for short periods do not materially affect the temperature of the concrete 3 ft. or more from an exposed face, and that it takes some 4 or 5 days for the effect of continued high or low air temperatures to appear in the temperature of the concrete, at that distance from the face.

The results obtained from Thermometer No. 3 indicate that there is very little seasonal variation in the temperature of the concrete 20 ft. or more from an exposed face.

The slight rise in temperature recorded by Thermometers Nos. 1, 3, and 4, about August 1st, and shown on the lower half of Plate VIII, tend to indicate that there may be a slight seasonal variation in all the masonry in the dam; however, the results obtained to date are insufficient to be conclusive on this point.

For an investigation of this sort, a long time is required before the drawing of many definite conclusions is warranted. Most of the opinions expressed in this paper seem to be warranted by the facts, but it is realized that the effect of chemical action in the concrete,

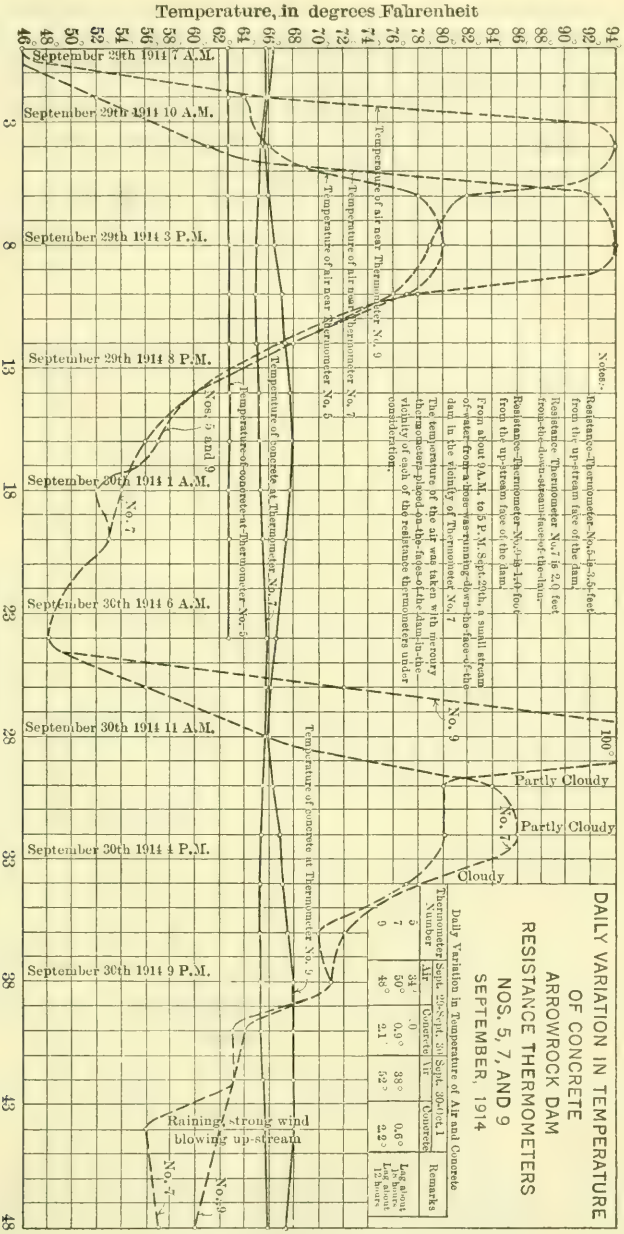

Temperature, in degrees Fahrenheit
Time, in Hours
DAILY VARIATION IN TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETERS NOS. 5, 7, AND 9 SEPTEMBER, 1914
Notes.-
Resistance-Thermometer No. 5 is 5.6 feet from the up-stream face of the dam.
Resistance Thermometer No. 7 is 2.0 feet from the down-stream face of the dam.
Resistance-Thermometer No. 9 is 1.4 feet from the up-stream face of the dam.
From about 9 A.M. to 5 P.M. Sept. 29th, a small stream of water from a hose was running down the face of the dam in the vicinity of Thermometer No. 7.
The temperature of the air was taken with mercury thermometers placed on the faces of the dam in the vicinity of each of the resistance thermometers under consideration.
Temperature of air near Thermometer No. 9
Temperature of air near Thermometer No. 7
Temperature of air near Thermometer No. 5
Temperature of concrete at Thermometer No. 7
Temperature of concrete at Thermometer No. 5
Temperature of concrete at Thermometer No. 9
Nos. 5 and 9
No. 7
No. 9
No. 7
Partly Cloudy
Partly Cloudy
Cloudy
Raining, strong wind blowing up-stream
No. 7
No. 9
September 29th 1914 7 A.M.
September 29th 1914 10 A.M.
September 29th 1914 3 P.M.
September 29th 1914 8 P.M.
September 30th 1914 1 A.M.
September 30th 1914 6 A.M.
September 30th 1914 11 A.M.
September 30th 1914 4 P.M.
September 30th 1914 9 P.M.
Daily Variation in Temperature of Air and Concrete
Thermometer Number
Sept. 29th-Sept. 30th
Sept. 30th-Oct. 1
Air
Concrete
Air
Concrete
Remarks
5
34°
.0
7
50°
0.9°
38°
0.6°
Long interval
9
48°
2.1
52°
2.2°
Long interval 12 hours

Fig. 8.

which is still going on, may not be fully understood, and that possibly later developments will show the need of modification of some of these conclusions.

It is hoped that there will be a full discussion of this subject, and that suggestions will be made for other experiments that may be carried on with the apparatus installed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE DESIGN OF HYDRO-ELECTRIC POWER PLANTS

By J. D. Galloway, M. Am. Soc. C. E.

To be Presented May 19th, 1915.

Synopsis.

The object of this paper is to examine certain of the problems relating to the design of hydro-electric power-plants, from the standpoint of the engineer in charge of the entire plant. The paper deals with hydraulic problems, and refers only incidentally to the electrical features. The detailed design of water-wheels is also omitted.

The particular features treated are: (1) the general conditions affecting the design of plants; (2) the conduit; (3) the penstock pipes; and (4) the station design.

In this examination, reference is made to the following points:

The load factor and its influence on design;

The relation of an auxiliary steam plant to the entire system, supplementing the work of the hydro-electric plant;

The rules of economy governing the design of all parts transmitting and transmuting energy;

The design of water conduits and regulating reservoirs, with data on the friction factors of flumes, and examples of conduit design;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The design of penstock pipes, with tables of the weight of riveted pipes and the auxiliary parts, such as valves, etc.;

The number of units in a plant, considering small stations, large stations, and stations as parts of a system;

The limitation of water-wheels and generators;

The general relations of head, power, and speed, with derivation of formula for specific speed;

The limitations of impulse-wheels, their field of use, examples of installations, and the value of the specific speed;

The limitations of turbines, their field of use, examples of installations, and the value of the specific speed;

The selection of a water-wheel and generator for different conditions;

Various curves illustrating the limitations of water-wheels;

Some general remarks on station design and the arrangement of auxiliaries.

The conclusions of the paper are as follows:

The load factor has a very important bearing on the design. A reserve steam station is necessary at the receiving end of a long line, and should be operated over the peak, in order to raise the load factor, at the hydro-electric station, up to an amount of 60 or 75 per cent. This will reduce the installation cost of the hydro-electric plant and transmission line in an amount sufficient to build the steam plant.

The rules of economy give results which can be only guides to an intelligent design of the energy conduits. Hence, judgment must largely govern the slope of water conduits, the size of penstocks, and the weight of copper in the line.

A regulating reservoir is necessary, in high- and medium-head plants, at the end of any free flowing conduit. The surge chamber on a closed conduit is to be resorted to only in the absence of a regulating reservoir.

The number of power units in a small station should be at least three, and in any station not more than four or five, unless the total available power is greater than can be generated by five units of maximum size possible for the particular case.

Impulse-wheels should preferably be limited to one nozzle on one wheel and to a value of 15 of the ratio of pitch diameter of wheel to

diameter of jet, giving a specific speed of 3.48, maximum, under normal working conditions.

Turbines have a present maximum limit of specific speed of 82.3 (366 metric) and a minimum limit of 9.4 (42 metric), but future designs may enlarge these limits.

Impulse-wheels are the only available motor for heads of more than 1 000 ft. (305 m.). Turbines, except for very small wheels, are the only motor for low heads, 200 ft. (61 m.). Between these limits, both wheels are available, with the tendency to use turbines on higher and higher heads as the size of the unit increases. This tendency is restricted by limitations of generator speed.

The limit of power developed by one unit has not been reached, and larger units may be expected in the future. The largest single hydro-electric generator is the 17 500 k.v.a. unit at Big Creek, Cal.

Introductory.

This paper is a review of some of the problems which arise in the design of hydro-electric power plants. It is limited to a discussion of the hydraulic features, and no reference is made to the electric parts, except in so far as they affect the design of the hydraulic elements. In the same manner, there is no discussion of the detailed design of water-wheels. The design of separate parts of a plant, such as generators, water-wheels, the pipe line, etc., has been treated in the publications of the various societies.

The object of this paper is to present the problem of design from the standpoint of the engineer who has to consider the entire plant. Due to the nature of the work and to the segregation of engineering into three separate branches, Civil, Mechanical, and Electrical, all of which have to do with hydro-electric plants, there has been considerable discussion of the separate parts of such plants in the publications of the various societies, very often with little consideration of the relation of the machine to other parts of the plant.

The engineer whose province is to assemble the entire plant into a working whole is necessarily without the detailed knowledge possessed by the engineer who designs the turbine or the generator, and must rely on the latter for such information. On the other hand, it often happens that the engineer of one of the special parts is not in com-

munication with the engineer who builds another part, which in the completed plant is directly related thereto. The turbine designer may set forth requirements, say of speed, necessary from his standpoint, which the generator builder cannot meet. It is hoped that this paper may bring out discussion of some of the questions involved, and by each of the three divisions of those interested in hydro-electric plant design.

It is not deemed necessary to refer to the development of water supply and the storage of water, for though such problems are an integral part of power plants, their discussion would extend the paper beyond proper limits and divert attention from the points on which it is desired to dwell.

Hydro-electric developments cover ranges in head of water from 10 up to 3 000 ft. or more. Although there are many differences of detail, the essential features are the same in all cases. Some of the problems of high-head plants are absent from those of low heads, and this is the excuse for dwelling more at length on the subject of high-head design. The particular features treated herein are: (1) the general conditions affecting the design of plants; (2) the conduit; (3) the penstock pipes; and (4) the station design.

General Conditions Affecting Plant Design.

The Load Factor.—Before taking up the subject of the design of parts, consideration must be given to the load factor. By this term is meant the ratio between the average power demand and the maximum or peak demand during any period in which a cycle of events is gone through, generally a day. A lighting load factor may be from 15 to 25%, that of a street-car system 50%, and that of a mill or a mine, running night and day, 80 to 90 per cent. A large power system has a factor which is the composite of all its separate load factors. Such factors vary considerably, as shown by Table 1.

TABLE 1.—Load Factors.

Niagara Falls Power Company	81.0%
Commonwealth Edison Company, Chicago	40.0%
New York Edison Company	34.7%
Philadelphia Electric Company	34.4%
Boston Edison Company	32.5%
Pacific Gas and Electric Company, California	59.0%
Great Western Power Company, California	70.0%
Sierra and San Francisco Power Company, California	50.0%
Pacific Light and Power Company, California	47.8%
Los Angeles Gas and Electric Company, California	40.0%

In order to emphasize some points of design by examples, a load factor of 50% will be chosen for this paper. A load curve of a street-car system producing this load factor will also be used.

The load factor affects all parts of the design, and is the governing condition in many cases. The water at an elevation contains a certain amount of energy, static when in the reservoir. This energy is carried by the water at first through canals and flumes, or by the natural bed of the stream, is transmuted into electric energy at the power-station, and in this form is transmitted to a distance by the lines. In all these various conduits containing and conducting the energy, the size and resulting cost is a direct function of the maximum amount of energy transmitted at any one time. Hence it is that the maximum demand fixes the size and cost of the conductors. As the average flow of energy is a measure of the resulting revenue, the load factor is of vital importance in the design, as it may be shown that a projected development may be unprofitable if the load factor is too low.

The possibility of using a reserve steam station to carry the peak of the power load should also be considered. On Fig. 1 is shown a typical load curve of a street-car system, the changes in the curve being taken by hours. The load factor of the curve is 50% and the power developed at any one time is shown as a percentage of the peak load. If the peak load is carried by the hydro-electric station, the entire equipment must be designed with reference to the maximum output. If, on the other hand, a reserve steam station is installed for emergencies, then use can be made of this station to reduce the peak on the hydro-electric station, or, in other words, to raise its load factor.

By allowing the steam station to take 40% of the system peak, the hydro-electric station must carry 60% of the peak. In this case the steam station generates some of the energy, and if a given quantity of water is available for the hydro-electric station, the total energy delivered by the two plants is increased over that of the hydro-electric station by the small amount generated at the steam station during peak load. On the curve in question, the hydro-electric station, by carrying 60% of the system peak, gains a load factor of 75% and at the same time generates 90.6% of the total

energy. The steam station carries 40% of the peak, has a daily load factor of 11.75%, and generates 9.4% of the total energy.

In such a system as this, if the hydro-electric plant has an average capacity of 15 000 kw., the peak load on a load factor of 75% is 20 000 kw.; but if the load factor is reduced to 50%, as given for the entire system, the peak load becomes 30 000 kw. This would require, in the latter case, an addition of 10 000 kw. of installation more than that necessary for a load factor of 75 per cent. As this

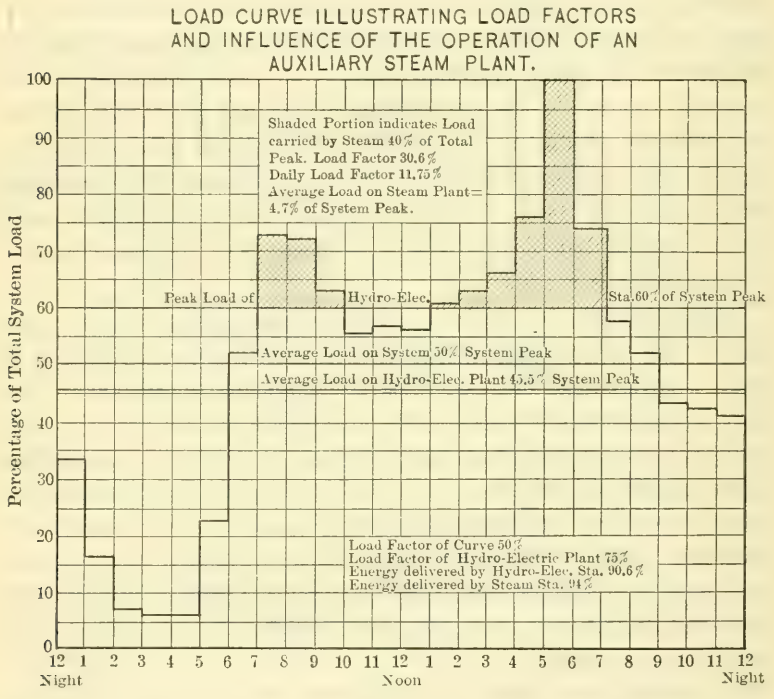

LOAD CURVE ILLUSTRATING LOAD FACTORS AND INFLUENCE OF THE OPERATION OF AN AUXILIARY STEAM PLANT.
Shaded Portion indicates Load carried by Steam 40% of Total Peak. Load Factor 30.6%
Daily Load Factor 11.75%
Average Load on Steam Plant = 4.7% of System Peak.
Peak Load of Hydro-Elec. Sta. 60% of System Peak
Average Load on System 50% System Peak
Average Load on Hydro-Elec. Plant 45.3% System Peak
Load Factor of Curve 50%
Load Factor of Hydro-Electric Plant 75%
Energy delivered by Hydro-Elec. Sta. 90.6%
Energy delivered by Steam Sta. 9.4%
Percentage of Total System Load
100
90
80
70
60
50
40
30
20
10
0
12 1 2 3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12
Night
Noon
Night

FIG. 1.

addition of installation affects the design and size of pipe lines, water-wheels, generators, transformers, and transmission lines, a large difference in cost is represented by the difference of 10 000 kw. At $100 per kw., this would amount to a difference of $1 000 000, which would build, under modern conditions, a steam plant of 20 000 kw., or as large as would be necessary in such an installation, as an emergency plant. The additional cost of operation, over the standby charges, would be slight, and is represented practically by the fuel.

For this there is compensation in the sale of the steam-generated energy.

These statements would be varied by every different kind of load curve, but in general they remain true. Depending on the form of the curve, and with present fuel prices, it can be shown that when a hydro-electric plant, with an ordinary length of transmission line, requires (for continued operation) to be supplemented by a steam plant, it is the best practice to operate the steam plant over the peak and raise the load factor on the hydro-electric plant up to the neighborhood of from 60 to 75 per cent.

A further requirement for raising the load factor at the hydro-electric plant is derived from the operation of the transmission line. The energy transmitted varies directly as the current, and the electrical losses vary as the square of the current. On a load factor of 50%, the energy transmitted at peak load is twice that at average load, at which time the losses would be approximately four times that of average load. On a 75% load factor, the losses at peak would be less than twice those at the average.

This entire subject becomes one where the conditions surrounding each particular case must govern. It is mentioned here as a major point affecting the design of hydro-electric plants.

The Rules of Economy.—The first rule of economy is that, in any given installation, the design should be governed by the consideration that the amount of the investment should be the least possible within the limits of good construction, other things being equal. In some cases where there is an abundance of water, it is not necessary to strive for the highest efficiency of the various parts, as the losses are not vital. This statement is true during a large part of the year in most plants, but, as a general rule, the power developed is measured by the minimum flow of water, and hence the economy of design holds true.

The second rule of economical design can be stated as follows: Any conduit carrying energy satisfies the requirement of minimum cost when the sum of the annual cost of conduit plus the value of the energy annually lost is a minimum. This rule governs all the conduits of the energy: ditches, flumes, tunnels, pipes, and transmission lines. As will be shown later, considerable latitude is possible in any

design, without departing far from the rule, and here the governing factor is to reduce the cost of the investment to a minimum.

The Value of Power.—In applying the rule of economy, the value is not that for which it is sold. It is the cost alone that measures the value as far as design of parts is concerned. If the design is made on this basis, it will be properly proportioned to the actual sale value of the energy.

The value of the energy at any place is a variable, depending on the position it then occupies in the course of its transmission from the source to the place of use. Judged by cost, the energy in water as it falls has no value. At the intake of the conduit it has a value depending on the value of water rights, cost of storage dams, and the diverting dam. At the end of the conduit the value of the contained energy has been increased by the cost of the conduit and regulating reservoir. At the high-tension bus-bars the value has been further increased by the cost of penstocks and power-station equipment, and, at the delivery end, there has been added the cost of the transmission line and the sub-station. A corollary of this proposition is: if money is to be spent to save energy by reducing losses, it should be apportioned, not along the course, but near the receiving end. If a given sum of money will save a given amount of energy otherwise lost, it is much better to apply it at the receiving end. If applied near the origin, say on the conduit, the energy saved must be transmitted through all the other parts of the system along the entire course. It must also be borne in mind that the amount of energy conveyed grows constantly less by losses, and is decreased to about 60% of the original amount at the receiving end.

In practice it is impossible to know the cost before the design is prepared, and assumptions must be made. If the cost of an installation is such that the resulting cost of energy amounts to 0.5 cent per kw-hour, the values may be apportioned at 0.2 cent for the conduit calculations, 0.3 cent for the penstocks, and 0.4 cent for the transmission line. Such values will give results of sufficient accuracy.

The Conduits and the Regulating Reservoir.

In many low-head plants, where the quantity of water used to develop a given amount of energy is large, the installation is limited in maximum output by the natural flow of the stream, supplemented

by such pondage as may be obtained by flash-boards or by the dam itself. In plants with high heads, where the water is brought a considerable distance in a conduit, the quantity of water is much less for a given development of power, and hence it is possible to control the flow delivered to the wheels by a regulating reservoir. If such a reservoir is available, the varying demand for water can be supplied from it, and the conduit can be designed for the average flow of water. In fact, in any open conduit of considerable length, it is hardly possible to regulate the flow at the intake to correspond to the varying flow at the power-station. In the case of closed conduits, such as pipes or pressure tunnels, surge chambers of proper design will obviate the difficulties of operation with a long penstock, but will not eliminate the necessity of designing the conduit for the maximum demand, thus largely increasing the cost. The regulating reservoir has the merit of reducing the cost of the conduit, of performing all the functions of a surge chamber, and, if large enough, furnishes an emergency supply of water when the conduit may be out of service. In these respects, it is far superior to a surge chamber, which should only be used when nothing else is feasible.

The Conduit Design.—The conduit may be either open or closed. The open conduit may be built of flumes, open canals, pipes crossing canyons, or tunnels. The formulas of Kutter or of Bazin may be used in determining the section, but that of Kutter is in most general use in America. For canals in earth, the coefficients given by Kutter may be used. If canals are lined with concrete, loss of water is prevented and the section is reduced by decreasing the friction. A value of $n = 0.014$ is believed to represent the concrete-lined canal or tunnel after it has been in use, although the value is lower in new, clean canals. For flumes built of surfaced lumber, with battens covering the cracks, a value of $n = 0.013$ to 0.014 is conservative. As there are few published data on large flumes, there are given in Table 2 the results of gaugings of two flumes made in 1906 by Charles D. Marx, President, Am. Soc. C. E., Mr. L. M. Hoskins, Wynn Meredith, M. Am. Soc. C. E., and the writer, preparatory to the design of the 14-mile flume of the Sierra and San Francisco Power Company. The Floriston flume had been in use about 10 years. It was 7 ft. 6 in. deep and 10 ft. 3 in. wide, built with surfaced lumber and rough § by 4-in. battens over the cracks. The

Fleish flume was nearly new, 6 ft. 0 in. deep, 10 ft. 0 in. wide, built with surfaced lumber, and the cracks were closed by a tongue, so that there were no obstructions in the flume.

TABLE 2.—Gaugings of Flumes.

	Floriston Flume.			Fleish Flume.
	Full flume.	Full flume.	Half flume.	Full flume.
1906	July 7th	July 9th	July 9th	July 8th
Length	600 ft.	300 ft.	300 ft.	500 ft.
Width	10.23 ft.	10.23 ft.	10.23 ft.	10.07 ft.
Depth	7.58 ft.	7.05 ft.	5.46 ft.	6.08 ft.
Number of battens	16	16	14	None.
Net area	77.21 ft.	71.79 ft.	55.57 ft.	61.23 ft.
Wetted perimeter	27.39 ft.	26.33 ft.	22.90 ft.	22.23 ft.
Hydraulic radius	2.819 ft.	2.727 ft.	2.427 ft.	2.754 ft.
Slope	0.00062	0.00064	0.000397	0.000426
Mean velocity	5.45	5.62	4.37	5.12
Q, in cubic feet per second	420.8	403.5	242.8	313.5
$c = \frac{v}{\sqrt{rs}}$	130.4	134.5	140.8	149.5
n in Kutter's formula	0.0135	0.0130	0.0125	0.0117

The shape of the cross-sections of the canals will depend on the material in which they are excavated, and also on the slope of the hillside. For economy in excavation, canals in level ground, or ground with slight slope, should be relatively wide and shallow, but on ground of steep slope, they should be relatively deep and narrow. On steep slopes there should be a berm on the upper side in order to prevent slides of material into the canal, which, especially if the canals are lined, reduces the carrying capacity materially.

Economical Slope for Conduit.—To illustrate the method of calculation of an economical slope for a conduit, one was assumed which would carry 350 sec-ft. of water, and be lined with concrete. To facilitate calculation and to produce uniform results, the section of the canal was given dimensions which are functions of the depth; with these, the slopes necessary to carry the water, with different sections, were determined. The cost of the canal, in lengths of 1 000 ft. of different sections, was then estimated. The annual value of the power lost by 350 sec-ft. of water falling 1 ft., or 29.7 kw., was taken for four assumed unit values: 0.2, 0.3, 0.4, and 0.5 cents per kw-hour, giving total values of $520, $780, $1 040, and $1 300, respectively, for the power per kilowatt-year. Table 3 is made up of these data, interest on the cost being taken at 6 per cent.

TABLE 3.—Summation of Annual Cost of 1 000 Ft. of Canal at 6% Annual Interest, and Value of Energy Lost in Different Slopes at $17.50, $26.25, $35.00, and $43.80, per Kilowatt Year. 350 Sec.-ft. of Water.

Depth of canal.	Cost, 1 000 ft. of canal.	6% annual interest.	Fall in 1 000 ft.	Val. $17.50 Kilowatt-Year.		Val. $26.25 Kilowatt-Year.		Val. $35.00 Kilowatt-Year.		Val. $43.80 Kilowatt-Year.	
				Value, power lost.	Power plus interest.	Value, power lost.	Power plus interest.	Value, power lost.	Power plus interest.	Value, power lost.	Power plus interest.
4.8	$6 960	$418	1.53	$796	$1 214	$1 195	$1 613	$1 595	$2 013	$1 985	$2 403
5.0	7 590	456	1.19	619	1 075	929	1 385	1 238	1 694	1 545	2 001
5.2	8 060	484	0.99	515	999	772	1 256	1 030	1 514	1 285	1 769
5.4	8 590	515	0.79	411	926	616	1 131	825	1 340	1 025	1 540
5.6	9 000	540	0.66	343	883	515	1 055	687	1 227	858	1 398
5.8	9 530	572	0.55	286	858	429	1 001	572	1 144	715	1 287
6.0	10 050	603	0.46	239	842	359	962	479	1 082	598	1 191
6.2	10 450	627	0.39	203	830	304	931	405	1 032	508	1 135
6.4	10 820	650	0.33	171	821	257	907	343	1 003	429	1 079
6.6	11 480	689	0.27	140	829	210	899	281	970	351	1 040
6.8	12 120	728	0.23	120	848	178	906	239	967	300	1 028
7.0	12 470	748	0.20	106	854	160	908	213	961	267	1 015
7.2	12 950	777	0.18	93	870	140	917	187	964	234	1 011
7.4	13 500	811	0.15	78	889	117	928	156	967	195	1 006
7.6	13 780	826	0.14	73	899	109	935	145	971	182	1 008
7.8	14 170	850	0.13	67	917	101	951	135	985	169	1 019
8.0	14 920	896	0.11	57	953	86	982	115	1 011	143	1 039

The data in Table 3 are set forth graphically on Fig. 2. A corresponding set of calculations for a tunnel is shown on Fig. 3, where the section of the tunnel is given.

The minimum values of the sum of annual cost of conduit, plus annual value of power lost, are at the lowest points on the curves. It will be noted that the curves are very flat near the minimum value. This will allow of considerable variation from the determined economical slope, and this variation should be toward the steeper slope, with smaller cross-section and lesser cost, in order to reduce the investment.

Flumes.—As a rule, it is not possible to vary the section of a flume to suit the slight variations of slope. If the flume is in short lengths between divisions of the canal, it can be placed on the same grade. If the conduit is a long flume, a few calculated sections will determine the economical slope. Fig. 4 shows a form of flume common in western America. It is built with "boxes" 16 ft. long.

General Statements.—The foregoing rule of economy cannot always be applied. A favorable site for a diverting dam and the location

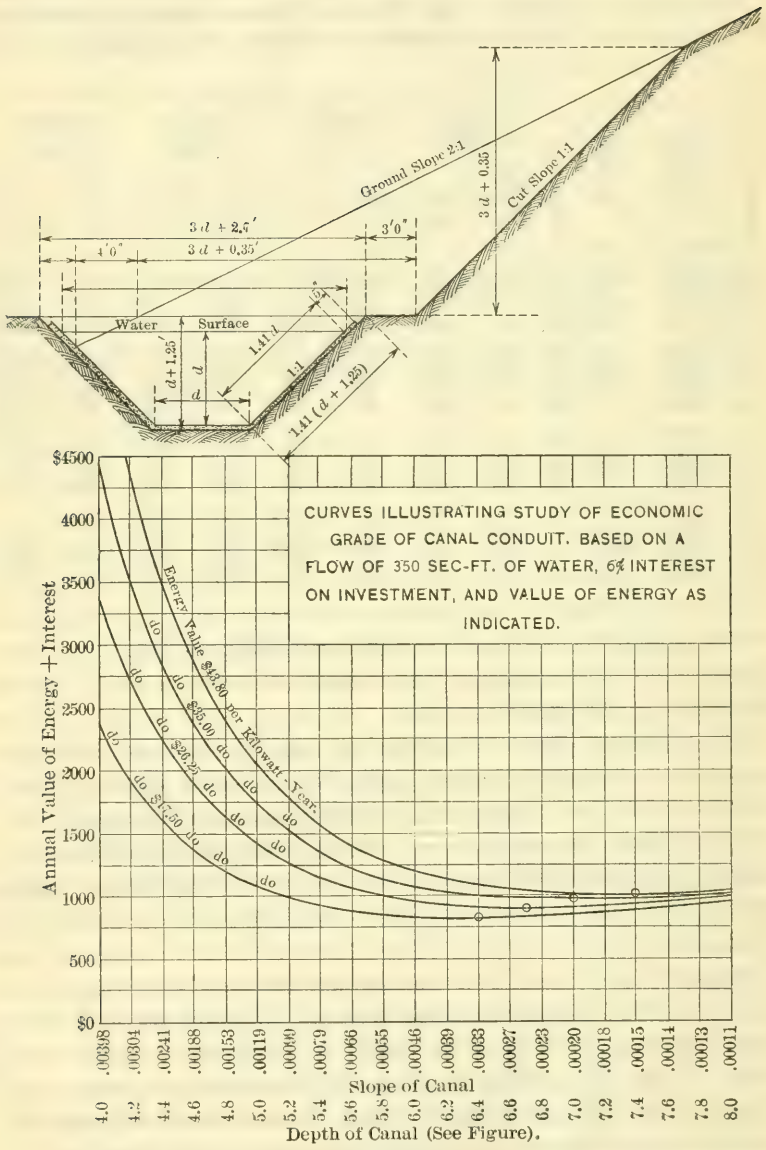
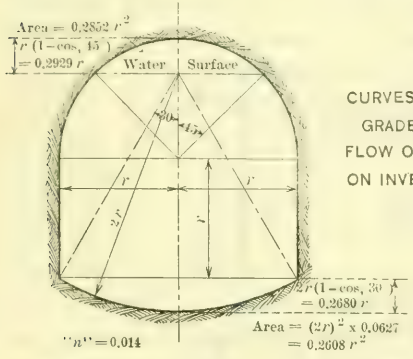

Ground Slope 2:1
Cut Slope 1:1
3 d + 2.7'
3 d + 0.35'
3'0"
4'0"
3 d + 0.35
Water Surface
1.41 d
d + 1.25
d
d
1:1
1.41 (d + 1.25)
CURVES ILLUSTRATING STUDY OF ECONOMIC GRADE OF CANAL CONDUIT. BASED ON A FLOW OF 350 SEC-FT. OF WATER, 6% INTEREST ON INVESTMENT, AND VALUE OF ENERGY AS INDICATED.
Energy Value $143.80 per Kilowatt-Year.
do $65.00 do
do $26.25 do
do $17.50 do
Annual Value of Energy + Interest
$4500
4000
3500
3000
2500
2000
1500
1000
500
$0
.00398 .00304 .00241 .00188 .00153 .00119 .00099 .00079 .00066 .00055 .00046 .00039 .00033 .00027 .00023 .00020 .00018 .00015 .00014 .00013 .00011
Slope of Canal
4.0 4.2 4.4 4.6 4.8 5.0 5.2 5.4 5.6 5.8 6.0 6.2 6.4 6.6 6.8 7.0 7.2 7.4 7.6 7.8 8.0
Depth of Canal (See Figure).

FIG. 2.

of a possible regulating reservoir often determine the grade of the conduit. In the absence of such controlling features, the rule of economy can be used merely as an intelligent guide in locating a conduit.



CURVES ILLUSTRATING STUDY OF ECONOMIC GRADE OF TUNNEL CONDUIT. BASED ON A FLOW OF 350 SEC-FT. OF WATER, 6% INTEREST ON INVESTMENT, AND VALUE OF ENERGY AS INDICATED.

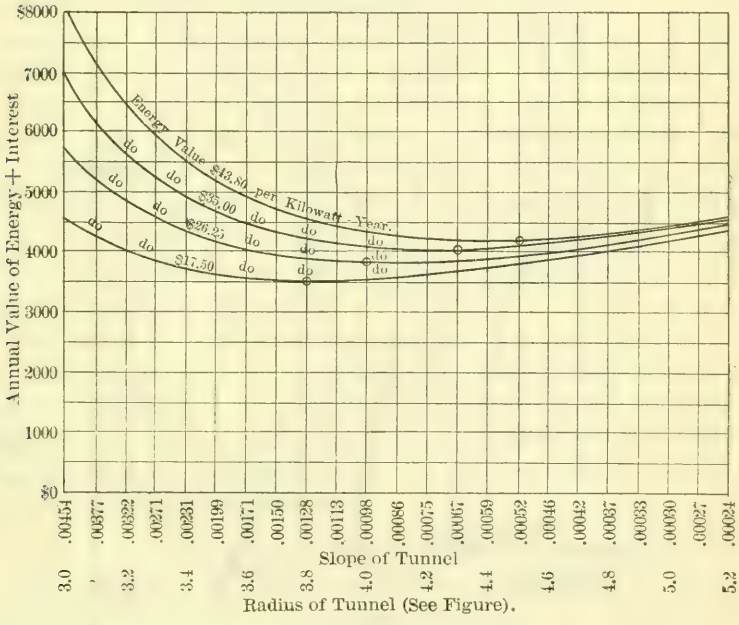


FIG. 3.

In the case of a closed conduit, consisting of a pipe or pressure tunnel, the economical section and slope would be determined in the same manner. The late Arthur L. Adams, M. Am. Soc. C. E., deduced the rule that a pipe satisfies the rule of economy when the

value of the energy lost in friction equals four-tenths of the annual cost of the pipe line.* This is merely a special case of the general rule. Other writers have also deduced rules for the economical diameter

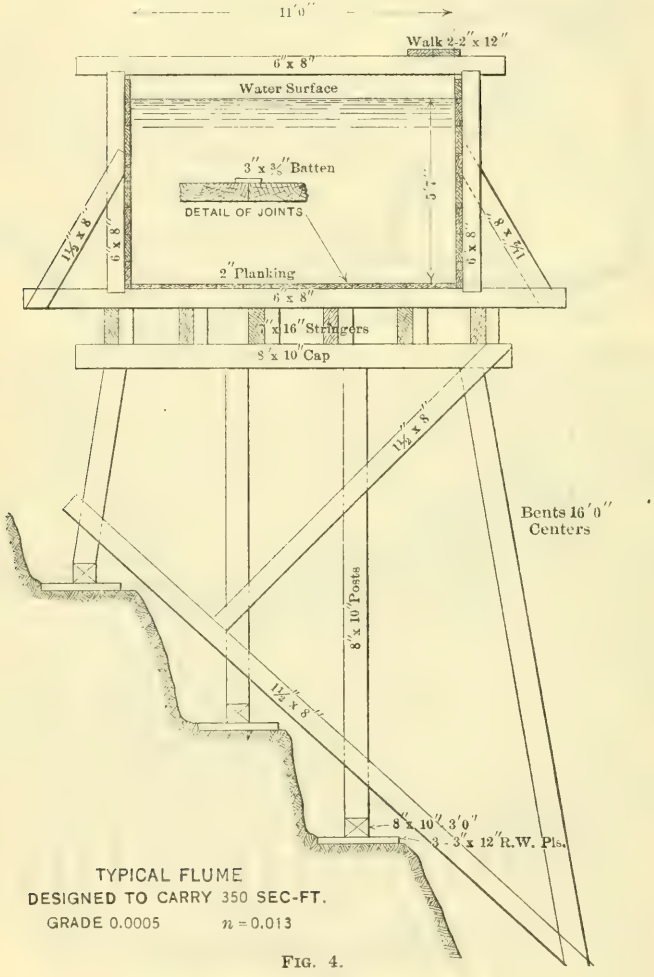


FIG. 4.

of pipes. In the application of these rules, the diameter of the pipe must be assumed, and, as the weight and resultant costs of pipe do not vary at a uniform rate with the diameter, it is believed that a

* *Transactions*, Am. Soc. C. E., Vol. LIX, 1907.

better way is to calculate a series of pipes in the manner illustrated herein, and determine which is the most economical in that way.

This disadvantage of the closed conduit, which is laid near the hydraulic grade line, lies in the necessity of designing the sections to suit the maximum demand, as a regulating reservoir is not possible. Thus, on a 50% load factor, the conduit would have to be of nearly twice the carrying capacity of one designed for a uniform flow, with resulting excess of cost. This, in many cases, is overbalanced by the additional head gained by connection with the storage reservoir. The fluctuations in the velocity of the water in the closed conduit, due to changes in power demand, can be cared for by surge chambers. The design of surge chambers is a problem involving a number of factors, and is not entered into here. The reader will find the subject fully treated in papers by Professor W. F. Durand,* of Stanford University, and Mr. R. D. Johnson.† The surge chamber is merely a special case of the regulating reservoir, without its two chief advantages, and should only be used where other solutions of the problem are impossible, unless an increased head is made available by the closed conduit. A distinction has been made herein between the conduit and the penstocks. The conduit has been assumed to be laid at or near the hydraulic grade line. The penstock commences at the end of the conduit and plunges directly down the incline to the power-station. The regulating reservoir, or the surge tank, is placed at or near the junction of the two.

The size of the regulating reservoir can be determined from the load curve by integration. As a matter of fact, if any reservoir is provided, an endeavor should be made to obtain one of a capacity sufficient to operate the plant over one day, in which case it has additional value as a source of energy in case of an emergency, such as a break in the conduit line.

The Penstock Pipes.

The installation of penstock pipes is necessary where the power-station is at a distance from the source of water. The determination of the economical size of the pipe can be made by selecting a number of possible diameters and calculating the best by the rule of economy.

* *Transactions*, Am. Soc. Mech. Engrs., Paper No. 1353, 1912.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, 1908; and *Proceedings*, Am. Soc. C. E., for October, 1914.

However, a number of factors must be considered in selecting the size of the pipe. The formula of Mr. Adams, for economical diameters, makes the tacit assumption that the diameter is constant throughout the length, which assumption is usually incorrect. If a given loss is assumed at the beginning of the calculations, it should be apportioned to different sections of the pipe on the profile of the ground. In such a case it will usually be found that limitations of manufacture will determine that a small diameter at the lower end and a large diameter at the upper end is the most desirable. Again, the limitation of a minimum thickness of plate will provide a pipe of large diameter at the top, thus allowing most of the total loss to be at the lower part of the pipe. Each problem is so special that it is advisable to lay out several pipes on a profile, calculate the costs and losses, and from these data determine the most economical pipe.

Other factors also affect the design. The flow of water in the penstock follows the load variations, and if the average flow is used to determine the size, then the losses at peak load may cause such a drop in pressure as to affect seriously the speed regulation. If the peak-load flow is used to determine the size, then the pipes may cost too much. The higher the load factor on the plant, the better will be the service from the pipes. Speaking generally, the peak-load losses should not exceed 10% of the total head, in which case, if the load factor is 50%, the average losses will be about 2½ per cent. With low load factors, the peak-load flow will determine the size; with high load factors, the average flow will govern.

The writer has a decided preference for riveted-steel pipes over the imported lap-welded pipe. As prices have prevailed in the West, the cost of lap-welded pipe is from 10 to 15% less than that of riveted pipe, the design taking account of different friction factors to produce the same carrying capacity. The several failures of the welded pipe are the cause of this opinion, and, as to the riveted pipe, there is no case of a properly designed pipe having failed.

In the design of riveted pipe, a factor of safety of four, based on strength of joint, is sufficient, provided the water-wheels are equipped with by-passes such that pressure rises of more than 25% are not possible. Medium steel is used, having an ultimate strength of from 60 000 to 65 000 lb. per sq. in. (4 218 to 4 570 kg. per sq. cm.). Double-riveted lap-joints of 70% efficiency, and triple-riveted butt-strap

joints of 80% efficiency, are possible. In the case of the 70% joint, the unit stress in the plates would be 11 200 lb. per sq. in. (837 kg. per sq. cm.), giving a safety factor in the plate of about 5.5%; and, in the case of the 80% joint, the stress in the plate would be 12 800 lb. per sq. in. (900 kg. per sq. cm.), giving a safety factor of about 4.75% in the plate. For 30-in. (76-cm.) pipes or greater, the writer is accustomed to use lap-joints up to a thickness of plate of $\frac{1}{2}$ in. (12.7 mm.), and butt-strap triple-riveted joints for greater thicknesses, and to use no plates less than $\frac{1}{4}$ in. in thickness. Figs. 5 and 6 give the static head which pipes of different diameters and thicknesses will stand. These are useful in preliminary layouts.

Table 4 gives the weight per foot of lap-riveted pipe, based on 6-ft. courses, the usual allowance for overweight of rolled plates, the calculated weights of the joints and rivets, and a covering of asphalt.

TABLE 4.—Weights, in Pounds per Foot, of Lap-Riveted Pipes.

Inside diameter, in inches.	Thickness of Plates.							
	$\frac{1}{8}$ in.	$\frac{3}{16}$ in.	$\frac{1}{4}$ in.	$\frac{5}{16}$ in.	$\frac{3}{8}$ in.	$\frac{7}{16}$ in.	$\frac{1}{2}$ in.	$\frac{9}{16}$ in.
18	33.9	49.5	64.7	78.6	93.6	109.9	126.7	147.0
20	37.4	54.6	71.0	86.5	102.7	120.5	138.6	160.7
22	41.1	59.7	77.6	94.3	112.1	131.1	150.9	174.4
24	44.5	64.4	83.8	102.0	121.1	141.7	162.8	188.0
27	49.9	72.2	93.7	113.9	135.1	157.8	181.0	208.4
30	55.3	79.8	103.3	125.3	148.7	173.5	199.1	228.8
33	60.8	87.6	113.1	137.2	162.7	189.7	217.4	249.4
36	66.0	95.1	122.3	148.7	176.5	205.4	235.4	269.9
39	71.3	102.5	131.6	160.3	190.4	221.5	253.6	290.2
42	76.6	110.2	141.8	172.2	204.0	237.4	271.5	310.7
48	87.3	125.3	161.0	195.5	231.8	260.3	307.9	351.9
54	98.0	140.6	180.2	218.9	259.2	301.1	344.1	392.8
60	108.8	155.8	199.5	242.5	286.9	333.0	380.4	433.9
66	119.4	170.8	218.7	265.6	314.5	364.9	416.6	474.6
72	130.0	186.1	237.8	288.8	342.0	396.7	452.8	515.6

Table 5 gives the weight per foot of triple-riveted, butt-strap pipe, based on 8-ft. courses, the usual allowance for overweight of rolled plates, the calculated weights of the joints, and a covering of asphalt.

In some of the thicknesses in the smaller pipes, it would be necessary to heat the plates before rolling.

It is desired to emphasize the necessity of placing air-valves on all pipes, as an emergency feature in case the water is suddenly drawn from the pipes. Many calculations have been made on the resistance

to collapsing of pipe against exterior pressure. Such calculations can never take into account all the conditions. The pipe is never a true cylinder, its own weight tends to distort it, and the water inside adds

SAFE STATIC HEAD FOR LAP-JOINT, DOUBLE-RIVETED, STEEL PIPE.

Maximum Stress = 11 200 Lb. per Sq.In. (787 Kg.per Sq.Cm.) Corresponds to a Stress in Plate of 16 000 Lb. per Sq.In. (1125 Kg. per Sq.Cm.) with a Joint of 70% Efficiency.

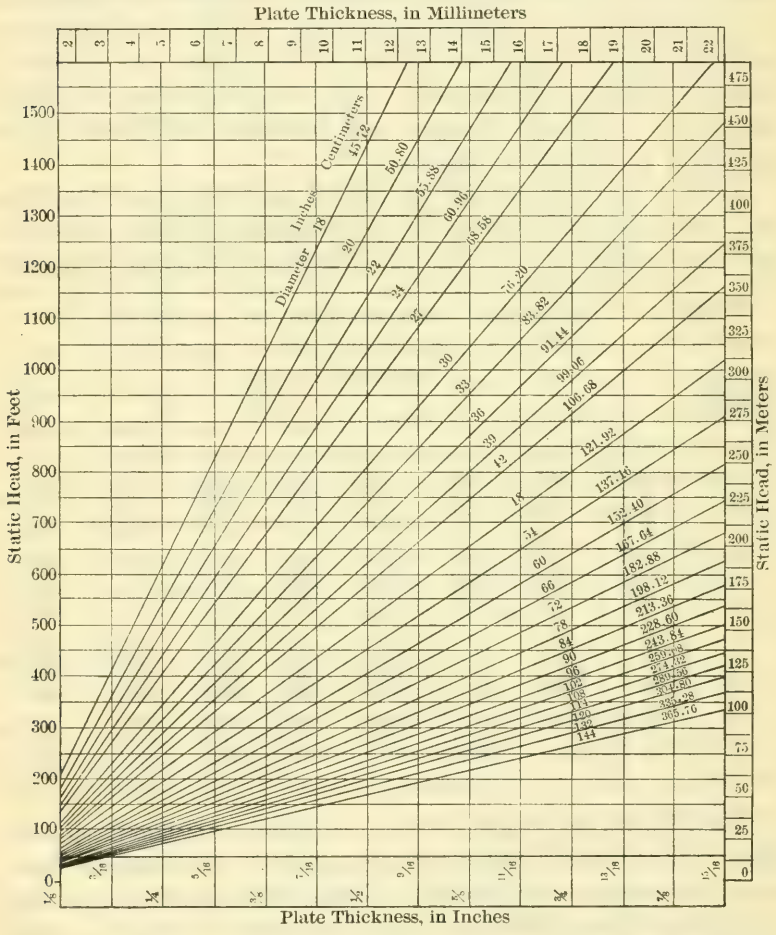


FIG. 5.

to this effect; and it is usually supported on piers and is acting as a beam. In the light of a number of disastrous failures, air-valves are believed to be a necessity on any pipe. They should be placed at high points, and, in long lines, not less than 500 ft. apart, and have

TABLE 5.—Weight, in Pounds per Foot, of Butt-Strap Riveted Pipes.

Inside diameter, in inches.	Thickness of Plates.								
	½ in.	9/16 in.	5/8 in.	11/16 in.	¾ in.	13/16 in.	7/8 in.	15/16 in.	1 in.
24	205.9	230.1	255.7	279.6	307.1	350.3	384.8	426.5	450.3
27	225.8	253.2	280.0	306.7	336.8	384.2	419.1	464.7	490.3
30	245.6	275.3	304.7	333.1	365.5	416.6	455.0	502.4	531.0
33	265.7	297.5	329.3	359.7	394.2	450.0	489.7	540.3	571.8
36	286.2	320.7	353.8	386.4	423.5	482.9	525.3	579.4	612.5
39	305.5	342.3	378.8	413.8	453.4	515.3	571.7	618.5	652.8
42	326.0	365.0	403.6	441.0	482.1	547.6	595.0	655.8	693.0
48	367.5	411.3	454.2	495.7	543.7	616.7	667.9	734.3	777.5
54	407.4	455.7	503.6	549.4	602.6	681.8	738.7	811.0	857.9
60	446.8	500.9	553.8	605.0	661.0	748.4	809.0	886.6	940.3
66	487.6	545.3	604.0	657.6	720.6	814.2	879.9	964.5	1 020.7
72	527.9	589.4	652.6	711.8	780.0	881.0	951.4	1 039.5	1 102.1

Inside diameter, in inches.	Thickness of Plates.							
	1 1/16 in.	1⅛ in.	1 3/16 in.	1¼ in.	1 5/16 in.	1⅜ in.	1 7/16 in.	1½ in.
27	524.4	566.0	.....	.....	.....	.....	.....	.....
30	567.0	611.9	653.1	686.4	.....	.....	.....	.....
33	609.9	657.8	702.5	736.6	776.1	809.2	.....	.....
36	654.1	704.9	751.3	788.2	831.8	864.9	908.8	949.9
39	690 3	750.9	799.2	838.1	884.1	920.4	964.8	1 009.9
42	739.3	796.3	846.5	889.4	936.1	976.4	1 025.2	1 071.3
48	826.6	891.2	946.9	994.5	1 046.9	1 091.6	1 144.7	1 195.9
54	912.0	983.7	1 044.7	1 096.8	1 153.5	1 204.4	1 260.9	1 318.5
60	1 004.9	1 076.1	1 142.2	1 199.6	1 261.2	1 318.2	1 380.9	1 443.7
66	1 086.5	1 169.1	1 239.3	1 301.7	1 369.5	1 427.1	1 494.8	1 561.7
72	1 171.6	1 260.6	1 336.1	1 401.7	1 473.5	1 539.9	1 612.9	1 682.4

an opening of at least 1% of the area of the pipe. They should not be less than 4 in. in diameter. There is no way of determining how fast the water may leave the pipe, for, as a rule, that occurs as the result of an accident. To assume a free flow of water out of the lower end would be too extreme a condition, as it is practically impossible. The foregoing statements are based on experience only. Plate IX shows a detail of an air-valve with gate-valve under, as placed on a recent installation. The gate-valve is placed so that the water may be shut off and the air-valve cleaned at intervals. Without the gate-valve, the air-valve is useless, as it soon "freezes" to its seat and will not act.

In the layout of pipe lines, there should be at least two pipes, and on high-head plants one pipe may be used to serve two generating units even of the largest size. For a four-unit plant, two pipes are

sufficient. In low- and medium-head plants, where the quantity of water is large, each generator should have its own pipe. At the junction with the conduit, the pipes should be provided with gate-valves, a cross-over with a gate-valve, and a stand-pipe or multiple-part air-

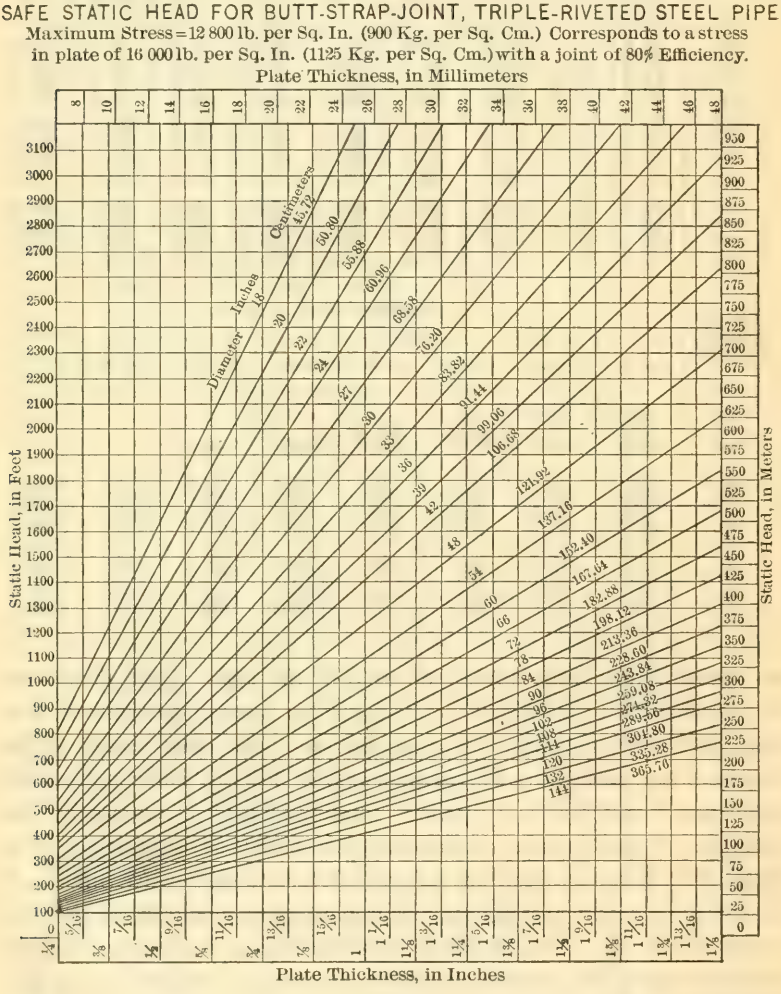
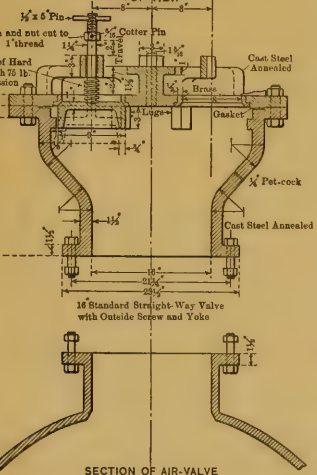
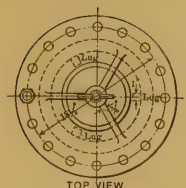
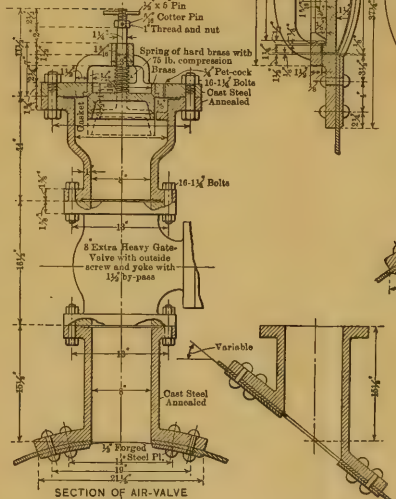

SAFE STATIC HEAD FOR BUTT-STRAP-JOINT, TRIPLE-RIVETED STEEL PIPE
Maximum Stress = 12 800 lb. per Sq. In. (900 Kg. per Sq. Cm.) Corresponds to a stress in plate of 16 000 lb. per Sq. In. (1125 Kg. per Sq. Cm.) with a joint of 80% Efficiency.
Plate Thickness, in Millimeters
Diameter, Inches
Centimeters
Static Head, in Feet
Static Head, in Meters
Plate Thickness, in Inches

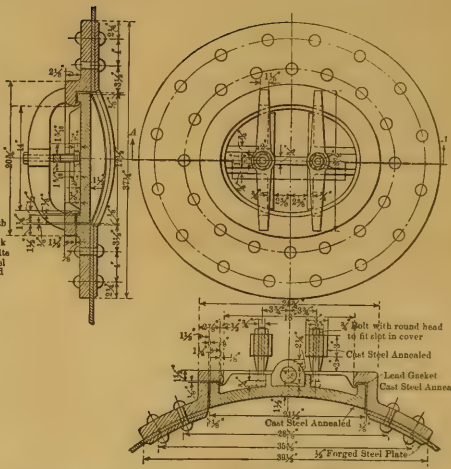
FIG. 6.

valve. The gate-valve should be hydraulically operated, if possible, and controlled from the power-house. On Pipe No. 5 of the Great Western Power Company, water for operating the gate-valve was obtained from a concrete water tank on the hill, 500 ft. above the valve,








¾" Bolt with round head to fit slot in cover
Cast Steel Annealed
Lead Gasket
Cast Steel Annealed
Cast Steel Annealed
½" Forged Steel Plate
SECTION OF MANHOLE ON "A-A"

DETAILS OF MANHOLES, AIR-VALVES, ETC.

water being supplied to the tank from the penstock by an electrically driven pump. At the power-station these pipes should be cross-connected, if very long, so that at off-peak loads, when only a few units are operating, advantage can be taken of all the pipe capacity. Such pipes should be provided with gate-valves at the power-house in such a way that one pipe can be isolated for repairs.

In some of the earlier stations it was customary to connect the penstocks to a header, usually at right angles, which extended for the length of the power-house and from which pipes led to the wheels. This practice has been discontinued in favor of pipes leading directly to the wheels, and if one pipe serves two or more wheels, branches are taken from it at small angles and led to the separate wheels.

Pipes on steep slopes should be supported on concrete piers and anchored thereto at frequent intervals, especially at vertical angles convex to the ground. They should have an extension joint near the upper end and, if possible, should be covered in the trench after test.

The Power-Station Design.

Some general statements may be made respecting the design of the power-station, but in this, as in other parts, the conditions under which the station is to operate will determine the form which the design will take. A station supplying an isolated district differs in layout from one which is merely a part of a large system.

The load factor of the system also has a bearing on the number of units in the station. A high load factor, 50% or greater, would require fewer units than a low load factor, as the more uniform load would not require changes in the quantity of generating apparatus in operation during the day.

Small Station Design.—The number of units in a station of an isolated system should be at least three, of which two would normally carry the load and one be considered a spare unit. If the system is provided with a steam reserve at the receiving end of the transmission line, of a capacity equal to or greater than one-half the full load, then the spare unit at the station may be omitted, if it is necessary to reduce the investment to a minimum. Such considerations give a minimum limit to the number of power units installed. For instance, if the total available power at the peak is 10 000 kw. and the load factor is high, three units of 5 000 kw. each could be installed, two

units to be in operation and the other to be considered a spare one. If the load factor is low, say from 25 to 30%, it would probably be better to install four 3 500-kw. units, three to be in operation and the other to be considered a spare one. The larger amount of installed capacity in the first case, 15 000 kw., would be largely offset in cost by the decreased unit cost of large units as compared with the smaller ones. Such considerations would govern in stations up to a total capacity of 30 000 kw. In such a case, with a low load factor, five 7 500-kw. generators might be preferable.

Large Station Design.—In the case of large stations, if the power to be developed on the isolated system is large, the number of units may be determined by the commercial sizes of the units available. Within reasonable limits, there is rarely need for more than four units in a station or, at the most, five, unless the total available power makes a greater number necessary. At the present time, units of 12 000 k.v.a. capacity and wound for 10 000 kw. at 0.8 power factor are common and may be considered a standard large-size unit. Units of 17 500 k.v.a. capacity are now in use in California. This in no way limits the possible size of units, and if the past is a gauge of the future, there will be a still further increase in size. Two years ago specifications were issued for a generator of 24 000 k.v.a. capacity operated by a 36 000-h.p. maximum rating turbine. Bids were received from responsible bidders for the construction, but it was not carried forward as it was deemed to be too much power in one unit in a station of 60 000 kw. capacity.

It may be said, however, that though there may be no structural or mechanical reason for limiting the size of generators, there may be a limit to the size due to operating conditions and the flexibility of the station. The size of the generator governs the size of the transformers and low-tension switches, and from such viewpoints as these, the concentration of a great amount of energy from one unit may be unwise. Some further consideration of this is given when referring to water-wheel design.

If the 10 000-kw. unit be named as the present standard, the number of units will be a function of the total power available, in any amount above 30 000 kw. If natural conditions reduce the size of unit that is possible, then in large plants there will be a number of the largest possible size. Thus, at Keokuk, Iowa, the plant of the

Mississippi River Power Company will contain thirty 7 500-kw. units.

Plants as Part of a System.—The foregoing remarks apply to isolated systems having one generating plant. It was noted that if a steam reserve is included in the system, the spare unit at the hydro-electric station might be omitted. When the contemplated plant will be in a territory where other plants are installed, it is often possible to make reciprocal contracts for emergency power, and this will allow the use of larger units; but this arrangement will hardly warrant the omission of a spare unit.

Where the contemplated plant is part of a large system having a number of generating stations, the unit may be of the largest size available, even to the total of the possible power, and no spare unit is necessary. The Deer Creek Plant of the Pacific Gas and Electric Corporation is an illustration of this plan. With ten hydro-electric plants of 123 650-kw. total capacity and four steam plants of 94 000-kw. capacity on the entire system, this station has one 5 500-kw. generator developing all the available power.

The Limitations of Water-Wheels and Generators.—In the foregoing comment on station design, no account was taken of the limitations imposed by the possible sizes of water-wheels. Two types of water-motors are available for operating generators: the impulse or Pelton wheel, and the hydraulic turbine, generally of the Francis type. The basic conditions of design limit the impulse-wheel to high heads and the turbine to low heads; on medium heads there is a zone where either type is available. The related questions of head, power, and speed control the design of each, and, with the restrictions on design of generators, there are determined certain limits which cannot be passed. All these have a bearing on the size of the unit, and necessarily modify any of the conclusions heretofore expressed.

Generators.—With few exceptions—and those are in Europe—the transmission of energy is accomplished by alternating-current electricity. The number of alternations per second, on different power systems, varies, being now either 25 or 60, with few exceptions, although in earlier work higher numbers of cycles were adopted. The three-phase system can also be said to be general. The number of cycles governs the number of poles and the speed of the generator, and hence, in any one system, the number of available speeds is definitely limited. In the discussion which follows, the 60-cycle system

is adopted, as one which is in general use and standard. The possible numbers of revolutions per minute, within given limits, are as follows: 150, 180, 200, 240, 300, 360, 400, 450, 514, 600, 720, 900, 1 800, and 3 600. Owing, also, to the necessity of keeping a practically constant speed on the generator, the head must be reasonably constant. This is readily obtained in high-head plants, but in low-head plants the range of head may be such as to require two wheels on a shaft, adapted to different heads.

General Calculations of Head, Power, and Speed.—In making comparison between water-wheels in different installations, and in the selection of wheels for a projected plant, use can be made of a relation which exists between head, power, and speed. This relation is obtained by developing the equation between unit quantities of head, power, and speed, which gives a result termed the specific speed. The specific speed is defined as that at which a water-wheel would operate if its dimensions were reduced to such an extent that it would generate 1 h.p. at 1 ft. head. The equation is derived in the following manner:

Let H = Effective head,
P = Power of the wheel,
D = Diameter of the wheel,
N = Speed, in revolutions per minute,
H_u = Unit head,
P_u = Unit power,
D_u = Diameter of homologous wheel which will develop unit power under unit head,
N' = Speed of given wheel under unit head,
P' = Power of given wheel under unit head,
N_s = Speed of homologous wheel under unit head and developing unit power, or specific speed.

The velocity of the water (and the corresponding velocity of the water-wheel runner) varies directly as $H^{\frac{1}{2}}$. Therefore,

$$\frac{N'}{N} = \frac{H_u^{\frac{1}{2}}}{H^{\frac{1}{2}}}, \text{ or } N' = N\frac{H_u^{\frac{1}{2}}}{H^{\frac{1}{2}}} \quad \ldots\ldots\ldots\ldots\ldots (a)$$

The power developed by any given runner varies directly as the quantity, Q, and the head. Hence, since Q varies as the velocity, which varies as $H^{\frac{1}{2}}$,

$$\frac{P'}{P} = \frac{H_u^{\frac{3}{2}}}{H^{\frac{3}{2}}}, \text{ or } P' = P\frac{H_u^{\frac{3}{2}}}{H^{\frac{3}{2}}} \quad \ldots\ldots\ldots\ldots\ldots (b)$$

In the homologous runner of unit diameter, the linear dimensions are directly proportional to those of the given runner, and hence the area of the passages of a turbine or the jet of an impulse-wheel vary directly as the square of the linear dimensions. Under a given head, the power varies directly as the area of the water passages. Hence

$$\frac{P'}{P_u} = \frac{D^2}{D_u^2}, \text{ or } D_u = D\,\frac{P_u^{\frac{1}{2}}}{P'^{\frac{1}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (c)$$

and

$$\frac{N'}{N_s} = \frac{D_u}{D}, \text{ or } N_s = \frac{D\,N'}{D_u} \quad \ldots\ldots\ldots\ldots\ldots\ldots (d)$$

Substituting in Equation (d) the values of N' and D_u,

$$N_s = \frac{D\,N\,\dfrac{H_u^{\frac{1}{2}}}{H^{\frac{1}{2}}}}{D\,\dfrac{P_u^{\frac{1}{2}}}{P'^{\frac{1}{2}}}} = \frac{N\,P'^{\frac{1}{2}}\,H_u^{\frac{5}{4}}}{P_u^{\frac{1}{2}}\,H^{\frac{5}{4}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (e)$$

when P_u = unit horse-power and H_u = unit head,

$$N_s = \frac{N\,P^1}{H^{\frac{5}{4}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (f)$$

This is a general formula for the specific speed, as first developed in Europe. Either the units of the foot-pound-second system or of the metric system can be used, but the numerical quantities will produce different numerical results. As the ratio of the foot to the meter is as 1 : 3.28, and the ratio of the horse-power to the metric horse-power is as 1 : 1.0139, by entering these units in the equation, it can be found that the specific speed metric is 4.447 times that in the foot-pound-second system.* The specific speed furnishes a method of comparison in the design of water-wheel runners, but that subject is outside the province of this paper. In addition, it provides a means

* The writer has followed the method of Mr. Chester W. Larner, in *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 306, in deriving the expression for N_s. There is considerable confusion in the use of this term. It was first applied to the quantity by European engineers, using metric units. Mr. Larner uses the foot-pound-second system of units, but designates the quantity "unit speed". Professor Zowski terms it the "type characteristic" and uses K_t to designate it. Professor Mead, in his work on "Water Power Engineering", uses the square of the quantity, designates it K_s, and names it "specific speed." It seems best to the writer to retain the name "specific speed", even with the use of the foot-pound units, as the relation is a general one and as N is in general use for designating the revolutions per minute, to designate specific speed by N_s. The subject might well be taken up by the American Society of Civil Engineers, and definite recommendations made as to the use of the term.

of comparing installations of water-wheels and discussion of limits of design.

Limitations of Impulse Water-Wheels.—In the development of impulse water-wheels in California, it was soon recognized that a definite relation existed between the pitch diameter of the runner and the diameter of the jet. This relation furnishes a limitation on the amount of power to be derived from one jet, the speed and head being a constant for any one installation. The following discussion is made to determine the limiting value of the specific speed under controlling conditions.

It can be shown that the maximum power is taken from the water when the peripheral velocity of the runner is one-half that of the jet. In practice, the ratio varies from 0.45 to 0.48 V. Experience has also shown that, in continuous operation, the ratio of diameter of wheel to diameter of jet should not be less than 15 to obtain the best efficiency, although less values have been used. In this way, the diameter of the jet establishes a relation between the speed, power, and head, the value of which can be obtained as a maximum value of N_s in the following way:

N = Speed, in revolutions per minute,
D = Pitch diameter of runner,
d = Diameter of the jet,
α = Ratio, $\frac{D}{d}$,
H = Effective head,
P = Power delivered to shaft = 0.8 of theoretical power in the water when efficiency of wheel = 80%,
V = Velocity of water at nozzle,
0.45 V = Velocity of periphery of wheel,
Q = Quantity of water at 62.5 lb. per cu. ft.

Then

$$V = \sqrt{2\,g\,H} = 8.021\,H^{\frac{1}{2}} \quad \ldots\ldots(1)$$

$$Q = \text{area of jet} \times \text{velocity} = \frac{\pi\,d^2}{4} \times 8.021\,H^{\frac{1}{2}} = 6.300\,d^2\,H^{\frac{1}{2}} \quad \ldots(2)$$

$$P = \frac{Q \times 62.5 \times H \times 0.8}{550} = 0.573\,d^2\,H^{\frac{3}{2}} \quad \ldots\ldots(3)$$

$$\text{Peripheral velocity of wheel} = \frac{\pi\,D\,N}{60} = \frac{\pi\,\alpha\,d\,N}{60} = 0.45\,V \quad \ldots(4)$$

But $$0.45\ V = 3.609\ H^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Therefore, $$3.609\ H^{\frac{1}{2}} = \frac{\pi\ \alpha\ d\ N}{60} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

or, $$d^2 = \frac{4\ 775\ H}{\alpha^2\ N^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

By substituting in Equation (3) the value of d^2,

$$P = \frac{2\ 725\ H^{\frac{5}{2}}}{\alpha^2\ N^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

If α is fixed at 15,

$$P = \frac{12.11\ H^{\frac{5}{2}}}{N^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

By reference to Equation (*f*), it is seen that the factor, 12.11, is the square of N_s, or $N_s = 3.48$, under the limiting conditions set forth.

Under such conditions, the maximum amount of power to be obtained from any one impulse-wheel with one nozzle can be determined when the head is known. As the speed of electric generators is limited as noted above, it is possible to construct diagrams showing the maximum possible power with different speeds of the unit. Fig. 7 shows a series of curves for ordinary possible generator speeds of 60-cycle current, from 150 to 900 rev. per min. for heads up to 2 200 ft. (670 m.) and power limits up to 28 000 h.p. (26 600 metric h.p.). The efficiency of 80% will represent average working conditions. Factors are also given by which the curve values may be multiplied to change the different values of N_s and $\frac{D}{d}$.

It is of interest to note the data of some actual high-head installations, for which purpose Table 6 is given. The horse-power is that of one nozzle on one wheel at maximum rating. At the maximum rating, the value of N_s, in several cases, exceeds that given by Equation (9), namely 3.48. As the normal power is generally 20% less than this, the given values of N_s in Table 6 should be reduced by 10.5% to obtain the value at normal output.

Limitations of Turbines.—The hydraulic turbine is subject to limitations in its design which affect the design of the combined hydro-electric unit. It is possible, however, to vary the design of the turbine runner in such a way that a wide range of conditions can be met and still retain the necessary efficiency in operation. With that

TABLE 6.—Data on Impulse Wheels.

No.	Location.	Size in horse-power.	Speed, in revolutions per minute.	Position of shaft.	Effective head, in feet.	Effective head, in meters.	N_s, in English units.	N_s, in metric units.
1	Adamello, Switzerland	8 000	420	Hor.	2 800	854	1.160	5.160
2	Arniberg, Switzerland	3 000	630	Hor.	2 800	854	1.510	6.720
3	Pacific Light and Power Company, Big Creek, Cal., No. 1	8 750	375	Hor.	1 900	570	2.989	13.320
4	Central Colorado Power Company, Boulder, Colo	10 500	400	Hor.	1 797	507	3.503	15.589
5	Tata Hydro-Electric Supply Company, India	8 000	300	Hor.	1 661	548	1.678	7.467
6	Sierra and Santa Fe Power Company, Stanislaus, Cal	6 000	400	Hor.	1 460	446	3.433	15.270
7	Pacific Gas and Electric Company, de Sabla, Cal., Nos. 1 and 2	3 700	240	Hor.	1 440	439	1.646	7.320
8	Pacific Gas and Electric Company, de Sabla, Cal., No. 3	7 500	400	Hor.	1 450	442	3.872	17.220
9	Pacific Gas and Electric Company, Electra, Cal., Nos. 1-5	3 700	240	Hor.	1 395	426	1.211	5.388
10	Pacific Gas and Electric Company, Electra, Cal., No. 6	7 500	400	Hor.	1 395	426	3.816	16.980
11	Pacific Gas and Electric Company, Drum Plant, Cal	10 000	360	Hor.	1 334	407	4.465	19.850
12	Necaxa Light and Power Company, Necaxa, Mexico	2 500	300	Vert.	1 300	397	3.6464	16.223
13	Pacific Gas and Electric Company, Electra, Cal., No. 7	7 500	400	Hor.	1 205	368	4.583	20.400
14	Los Angeles Aqueduct, San Francisquito No. 1	7 000	200	Hor.	905	276	3.371	15.000
15	Los Angeles Edison Company, Kern River, Cal., No. 1	5 375	250	Hor.	865	264	3.907	17.387
16	Pacific Gas and Electric Company, Deer Creek, Cal	3 700	300	Hor.	800	122	4.289	19.086

subject this paper does not deal. The design of turbine runners is still in a state of evolution. By experiment with model runners at Holyoke, American designers are producing runners with efficiencies up to 93%, and adapted to a wide range of conditions. The specific speed furnishes a method of comparison, and its maximum and minimum values are an index of the present limitations of turbines. Table 7 presents the data of some recent installations. Medium power refers to units up to 5 000 h.p., and high power above that limit. Low heads include those up to 200 ft. (61 m.), and high heads those above that limit. The power given is the maximum for one wheel.

A study of the data in Table 7 shows that for low heads the value of N_s in low-head plants, with medium-power wheels, varies from 48.1 (186 metric) to 75.7 (336 metric); and with low heads and high-power wheels, N_s increases. The highest value given is that of the Cedars Rapids Manufacturing and Power Company, on the St. Lawrence River, or 82.3 (366 metric). The value of 75.8 (337 metric) at Keokuk and 75.5 (336 metric) for the 20 000-h.p. wheels of the

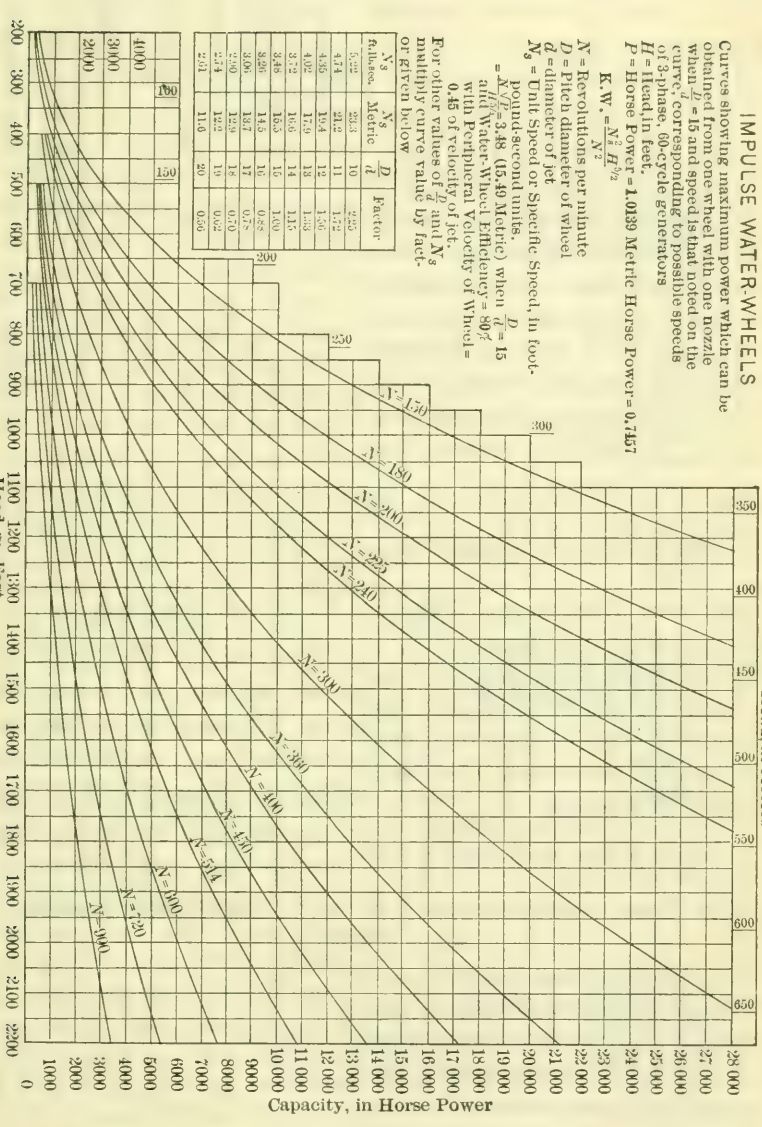

IMPULSE WATER-WHEELS
Curves showing maximum power which can be obtained from one wheel with one nozzle when $\frac{D}{d}$ = 15 and speed is that noted on the curve, corresponding to possible speeds of 3-phase, 60-cycle generators
H = Head, in feet.
P = Horse Power = 1.039 Metric Horse Power = 0.757 K.W. = $\frac{N_s^2 H^{5/4}}{N^2}$
N = Revolutions per minute
D = Pitch diameter of wheel
d = diameter of jet
N_s = Unit Speed or Specific Speed, in foot-pound-second units.
= $N\sqrt{P}/H^{5/4}$ = 3.48 (15.49 Metric) when $\frac{D}{d}$ = 15 and Water-Wheel Efficiency = 80%
with Peripheral Velocity of Wheel = 0.45 of velocity of jet.
For other values of $\frac{D}{d}$ and N_s multiply curve value by factor given below
N_s (Values, 5.22, 4.74, 4.35, 4.02, 3.72, 3.48, 3.26, 3.06, 2.90, 2.74, 2.61) | N_s Metric (29.3, 26.2, 24.4, 22.6, 20.9, 19.5, 18.3, 17.2, 16.3, 15.4, 14.7) | $\frac{D}{d}$ (10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20) | Factor (1.25, 1.172, 1.136, 1.103, 1.15, 1.00, 0.88, 0.78, 0.70, 0.62, 0.56)
N = 150
N = 180
N = 200
N = 225
N = 240
N = 300
N = 360
N = 400
N = 450
N = 514
N = 600
N = 720
N = 900
Head, in Meters
Capacity, in Horse Power
Head, in Feet

Fig. 7.

TABLE 7.—Data on Turbines.

No.	Location.	Size, in horse-power.	Speed, in revolutions per minute.	Position on shaft.	Effective head, in feet.	Head, in meters.	N_s, in foot-pound units.	N_s, in metric units.
Medium Power (Up to 5 000 h.p.). Low Head (Less than 200 Ft.).								
1	Washington Water Power Company, Little Falls	4 500	150	Hor.	66	20.12	75.7	336
2	Appalachian Power Company, Development No. 4	3 500	97	Vert.	34	10.35	69.9	310
3	Georgia-Carolina Power Company, Stevens Creek	3 125	75	Vert.	27	8.23	68.1	303
4	Schenectady Power Company, Schaghticoke	5 000	300	Vert.	146	44.50	48.1	186
5	East Creek Electric and Power Company, Little Falls, N. Y.	4 000	300	Vert.	115	35.05	50.4	224
6	Southern Power Company, Ninety-nine Islands	2 600	225	Hor.	72	21.95	54.7	243
7	Central Georgia Power Company, Ocmulgee River	2 750	300	Hor.	100	30.50	70.4	313
8	East Creek Electric and Power Company, East Canada Creek, N. Y.	4 000	300	Vert.	115	35.05	50.4	224
9	East Tennessee Power Company, Ocoee River	2 700	360	Hor.	98	29.85	60.7	270
Medium Power (Up to 5 000 h.p.). High Head (Greater than 200 Ft.).								
1	Telluride Power Company, Grace Station.	4 250	300	Hor.	450	128.00	9.4	42
2	Cleveland Cliffs Iron Company, Carp River	4 000	720	Hor.	580	176.70	16.0	71
High Power (Greater than 5 000 h.p.). Low Head (Less than 200 Ft.).								
1	Appalachian Power Company	6 000	116	Vert.	49	14.95	69.3	308
2	Alabama Power Company	17 500	100	Vert.	68	20.75	67.7	301
3	Cedars Rapids Manufacturing and Power Company, St. Lawrence River	10 800	55.6	Vert.	30	9.15	82.3	366
4	Mississippi River Power Company, Keokuk, Ia	10 000	57.7	Vert.	32	9.75	75.8	337
5	Alabama Power Company, Coosa River Lock No. 12	17 500	100	Vert.	68	20.75	67.7	301
6	Laurentide Power Company, Grand Mère, Canada	20 000	120	Vert.	76	23.20	75.5	336
7	Canadian Niagara Falls	10 000	250	Vert.	130	39.70	57.0	252
8	Central Colorado Power Company, Shoshone Plant	9 000	400	Hor.	170	51.80	61.8	275
9	Washington Water Power Company, Long Lake	11 250	200	Hor.	168	51.20	35.0	350
High Power (Greater than 5 000 h.p.). High Head (Greater than 200 Ft.).								
1	Georgia Railway and Power Company, Tallulah Falls	16 000	514	Vert.	580	176.7	22.8	102
2	Great Western Power Company, Las Plumas, Cal	18 500	400	Vert.	465	141.7	25.2	112
3	Pacific Coast Power Company, White River, Wash.	18 000	360	Hor.	440	138.2	24.0	107
4	Pacific Gas and Electric Company, Centerville, Cal	9 700	400	Hor.	565	172.0	13.5	61
5	Michoacan Power Company, Noriega, Mexico	6 000	514	Hor.	670	202.1	11.7	52
6	Seattle Municipal Plant, Cedar River, Wash	8 000	600	Hor.	584	178.0	18.7	83
7	Tacoma Municipal Plant, Nisqually, Wash.	8 000	450	Hor.	400	121.9	22.5	100

Laurentide Power Company, at Grand Mère, Canada, are next, and represent the most recent installations. It is probable that this limit will be exceeded as development continues, reaching a value of N_s of 90 (402 metric). Such values correspond to relatively low speeds in revolutions per minute, in which case the generator design is susceptible of a considerable variation on account of the large size of the unit.

In this paper more interest attaches to the design of turbines under relatively high heads, namely 200 ft. (61 m.) and greater. Here the value of N_s approaches a minimum. The range is from 9.4 (42 metric) at Grace, Idaho, up to 25.2 (112 metric) at Las Plumas, Cal. To set forth the limitations of turbine design in relation to power and speed, the curves on Fig. 8 were drawn, with a value of $N_s = 15$, the speeds of 60-cycle generators, and a range of head up to 1 000 ft. (305 m.). Opinions may differ as to whether the value of $N_s = 15$ is the proper limit, as a number of plants of higher head have values less than this. It is somewhere near the limit of design, when considering operating conditions, but a table of factors by which to multiply the results shown by the curve is given, by which they can be changed to correspond to other values of N_s.

It is to be noted that the curve values give the minimum quantity of power which a wheel will develop under given conditions of head and speed. To reduce the quantity of power reduces the value of N_s, and this may not be desirable on account of operating conditions, such as injury to the runner from the impact of water under part loads. Hence, at high heads, the problem becomes one where a properly designed wheel may develop more power than required.

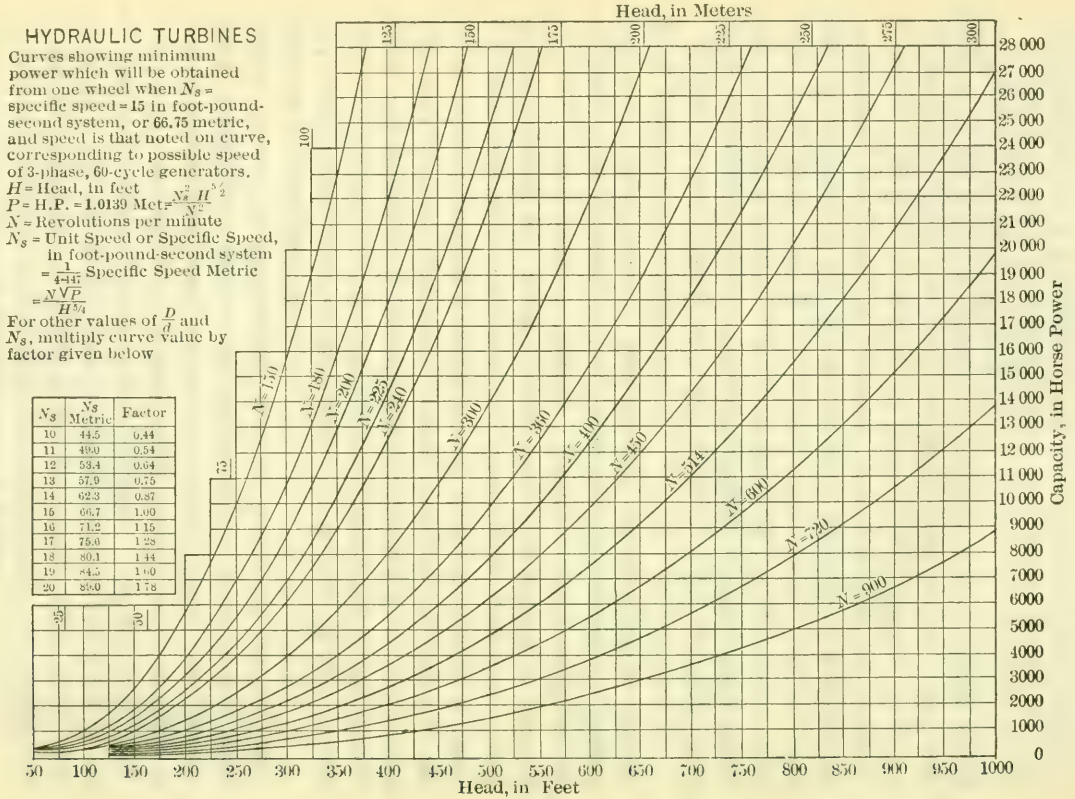
Selection of a Water-Wheel.—The selection of the type of wheel for heads varying from 200 to 1 000 ft. (61 to 305 m.) becomes thus a problem involving several factors. The impulse-wheel has a definite maximum of power beyond which it cannot go. Two wheels with one nozzle each represent the maximum power of good design, although more nozzles have been placed on one wheel. The high-power impulse-wheel requires a low speed, with resulting higher cost of generator. For heads greater than 1 000 ft. (306 m.), it can be said that at present it represents the only available type of water-motor, and the many high-head installations are a testimony to its value.

The turbine, on the other hand, is primarily the water-motor for low heads. The only trouble with turbines with a high value of N_s is that their efficiency is a maximum at a point near the full capacity, and that it drops off rapidly each way from that point. Hence, under partial loads, the recent high efficiencies obtained as a maximum do not hold. High-head turbines with a low value of N_s are not subject to this rapid change of efficiency.

The subject of efficiencies at partial loads is not so important as it would appear at first glance. During a large part of the year there is usually an excess of water, which, if not used, goes over the dam. Again, most modern stations have installed and in operation a number of units. All but one of the units in operation at any one time can be given full loads at the point of maximum efficiency, the fluctuations in loads being taken by a single unit.

Turbines, in addition to being from 5 to 7% more efficient than impulse-wheels, use a portion of the head by the draft-tube, which is lost with the impulse-wheel. A 20-ft. draft-tube on a head of 500 ft. (152.5 m.) will give a saving of 2.5% of the power. The disadvantages appear in the selection of a turbine under high heads. For a medium-power unit, the speed must be so high that it passes the limit of proper generator design. The speed of medium- or high-power generators seems at present to be limited to 360, 400, and 514 rev. per min. for 60-cycle machines. This can be exceeded, but the cost will be greater. If the speed is kept within limits, the value of N_s decreases, resulting in operating conditions which will cause wear on the runner. The use of high-speed generators results in a small machine and a lessened rotating moment, with resulting effect on the governing of the unit. A few examples will bring out the points mentioned.

Case 1.—Consider a generator of 4 000 kw. with a 25% overload, or 5 000 kw., with a generator efficiency of 95 per cent. This will require a wheel delivering 7 500 h.p. If the head is 300 ft. (91.5 m.), it is clearly beyond the range of two impulse-wheels with one nozzle each. A single-runner turbine with a speed of 225 rev. per min. or greater, will answer. If the head is 600 ft. (273 m.) a single-runner single-nozzle impulse-wheel, with a generator of 200 rev. per min., or two impulse-wheels with single nozzles, with a speed of 300 rev. per


HYDRAULIC TURBINES
Curves showing minimum power which will be obtained from one wheel when N_s = specific speed = 15 in foot-pound-second system, or 66.75 metric, and speed is that noted on curve, corresponding to possible speed of 3-phase, 60-cycle generators.
H = Head, in feet
P = H.P. = 1.0139 Met = $\frac{N_s^2 H^{5/2}}{N^2}$
N = Revolutions per minute
N_s = Unit Speed or Specific Speed, in foot-pound-second system = $\frac{1}{4.447}$ Specific Speed Metric = $\frac{N\sqrt{P}}{H^{3/4}}$
For other values of $\frac{D}{d}$ and N_s, multiply curve value by factor given below
Head, in Meters
Head, in Feet
Capacity, in Horse Power
N = 150
N = 180
N = 200
N = 225
N = 240
N = 300
N = 360
N = 400
N = 450
N = 514
N = 600
N = 720
N = 900

N_s	N_s Metric	Factor
10	44.5	0.44
11	49.0	0.54
12	53.4	0.64
13	57.9	0.75
14	62.3	0.87
15	66.7	1.00
16	71.2	1.15
17	75.6	1.28
18	80.1	1.44
19	84.5	1.60
20	89.0	1.78

FIG. 8.

min., will answer. Conditions of cost and governing would determine the design. If a turbine were used, the speed would be 900 rev. per min., which is beyond the present limits of design.

Case 2.—Consider a generator delivering 10 000 kw. normally, or 12 500 kw. overload, requiring 18 000 h.p. The lowest head under which two impulse-wheels with one nozzle each, and a speed of 150 rev. per min., would operate is 780 ft. (238 m.). At this head, a turbine of 514 rev. per min., with a single runner, would probably be better. On the other hand, at a head of 1 000 ft. (305 m.) the impulse-wheels could have a speed of 200 rev. per min., and the turbine would require a speed of 600 rev. per min., or better, 720 rev. per min., requiring a special generator.

It can thus be seen that the selection of the size of unit depends on many conditions, and that the choice is restricted to certain definite speeds and limited by certain conditions. For low heads, the turbine holds sway, and for high heads, greater than 1 000 ft. (305 m.), the impulse-wheel is the only available motor. Between these limits, then, is a space where the two overlap. The complex questions of cost, efficiency, design, and operation must also be considered in selecting the unit. The increase in efficiency of units has been dwelt on, and the recent improvements in American turbines is remarkable. It must be borne in mind, however, that the high efficiency can be effective during a portion of the year only.

There has been considerable discussion as to the merits of horizontal- as compared with vertical-shaft turbines. With wheels having a draft-tube to each runner, the accessibility of the unit with a horizontal shaft would seem to be the determining factor. On the other hand, if the floods of the stream rise much above the low water, the vertical-shaft type allows the generator to be placed at any necessary elevation. The free entrance of the water to and discharge from the vertical shaft unit is a point in its favor. The success of the step-bearings of the roller or Kingsbury type, for vertical shafts, makes this feature no longer a deciding factor.

It has been fairly well settled that each runner should have its own draft-tube, as a discharge of two wheels into one tube results in greater losses.

General Remarks on Station Design.—A few other points may be mentioned as affecting station design. Turbines should be provided

with relief valves, of a capacity to discharge at least 75% of the water passing the turbine, and direct-connected to the operating cylinders of the turbine. In the case of impulse-wheels, either the deflecting nozzle or a by-pass nozzle should be used. In both cases, the operation should be directly controlled from the governor.

In a station of several units, there should be, for the governors, a central system of pressure oil supply, with pumps in duplicate and pipes leading to the accumulator tanks at the governors. This system is believed to be preferable to the individual pump at each governor. If the station has vertical shaft units, there should be a similar central system of pressure oil for the step-bearing.

All power units used as auxiliaries should be driven by the primary source of power, the water. The exciter should be in duplicate, and, in addition to the water-wheels, one should have an electric motor to be used in emergencies. The pumps for the oil system can be driven by water, either by piston pumps, rotary pumps, or impulse-wheels. Gate- or pivot-valves back of the turbine should be operated by hydraulic cylinders. The basic reason for these statements is the fact that the auxiliaries are used to control the electric generators. They should be free from trouble at the time when the generator requires attention. If electrically driven, the auxiliaries merely reflect the generator trouble and add to it in emergencies.

All parts requiring cast metal, such as scroll cases of turbines, bodies and bonnets of gate-valves, and curves in pipes subject to water pressure, should be of annealed cast steel. Such parts should be of cylindrical or spherical section. The standard cast-iron gate-valve, with rectangular cross-section of the bonnet ribbed on the outside, is an extreme case of bad design. The ribs should be on the inside, if used at all.

The writer will venture on the electrical design at one point only. If the station is of fair size, say 20 000 kw., there should be a double set of bus-bars, with a complete set of switches for transferring the energy to the lines. Generally, one line takes the output of two or more generators, in which case considerable flexibility is necessary.

In endeavoring to keep within the prescribed limits of the subject matter, the writer may have failed to refer to a number of points which others may consider important. The subject is treated in a

general way, and though definite statements of opinion are advanced, it is recognized that there are many reasons for holding different views. If the paper serves to arouse discussion, especially on the subject of the proper water-motor to use under medium heads, it will have served the purpose for which it was written.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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THE PUMPING PLANT OF THE MORENCI WATER COMPANY

By W. L. Du Moulin, Assoc. M. Am. Soc. C. E.

To be Presented June 2d, 1915.

Synopsis.

The aim of this paper is to describe the pumping plant of the Morenci Water Company, and to relate operating experiences and difficulties encountered in its construction.

The plant is about 6 miles southwest of Morenci, Ariz., on the Eagle River. It furnishes all the water used for domestic purposes by the inhabitants of the Town of Morenci and also by the two copper mining companies operating there.

The town is an unincorporated mining camp in Greenlee County, in the middle of the rough, mountainous region of southeastern Arizona. Copper mining is the sole industry, and is responsible for the camp's existence. The two mining companies are The Detroit Copper Mining Company, owned by Phelps, Dodge and Company, operating a 1 300-ton mill and a 350-ton smelter; and The Arizona Copper Company, Limited, controlled by Edinburgh capital, operating a 3 000-ton mill. The smelter of this company is at Clifton, Ariz. The population of Morenci is about 6 000, 20% being American and 80% Mexican.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The physical character of the country surrounding Morenci, and in the direction of the pumping plant particularly, is exceedingly rough, rocky, and mountainous, and practically barren—in fact, almost a desert.

There is no water obtainable near the town. A small quantity was formerly pumped by the mining companies from the San Francisco River, some 9 miles distant, stored in small tanks, and carried around camp on the backs of burros, mules, and Mexicans, but it was very dirty and unwholesome. The Morenci Water Company was formed in October, 1898, to pump water for domestic purposes from the Eagle River. This water is distributed by gravity to the houses around camp through pipe lines from storage tanks, and is very good and wholesome. In 1908, a larger plant was needed to supply the increased demand for domestic purposes. At the same time, the mining companies decided to abandon pumping for their own use the bad water from the San Francisco River, and made arrangements with the water company for the better supply from the Eagle River. To provide the quantities required by the mining companies and for domestic purposes, the water company built the present plant, with its machinery of special design. This enlarged plant, with its efficient equipment, has thus replaced a number of small plants with inefficient equipment and high pumping costs, and has pumped the larger quantity of water at much less cost.

This is perhaps the only pumping plant in the United States furnishing water to a community for domestic purposes, where water is delivered in one lift through about 5 miles of pipe lines against a static head of 1 525 ft. It consists of three pumping engines, two being triple-expansion, condensing, high-duty engines, and one cross-compound, condensing, high-duty engine, with the necessary boilers, etc. The total capacity of the plant is 4 500 000 gal. per 24 hours.

The whole plant, together with the distributing system, etc., represents a valuation of $675 000.

Structures.

The pumping station contains a main engine-room, a smaller engine-room, a boiler-room, and an addition for miscellaneous reserve equipment, and covers an area of 176 ft. 0 in. by 78 ft. 6 in. The building proper is of substantial steel construction, consisting of a

FIG. 1.—MORENCI, ARIZ. THE TWO WATER STORAGE TANKS ARE ON THE HILL.

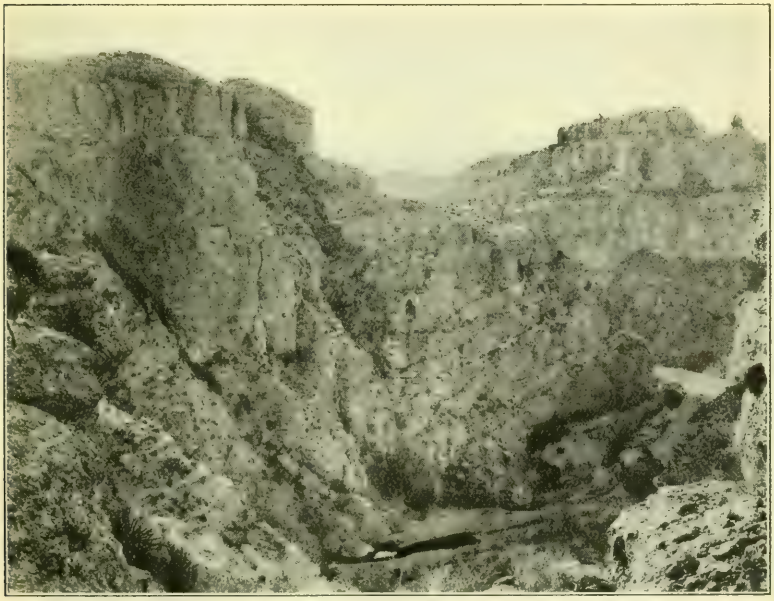


FIG. 2.—COUNTRY IN THE VICINITY OF MORENCI.

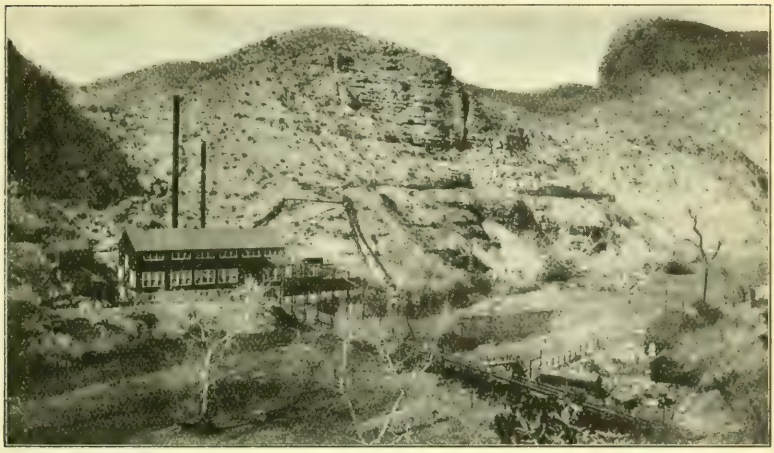


FIG. 3.—PUMPING PLANT, MORENCI WATER COMPANY. SETTLING BASINS IN THE DISTANCE.

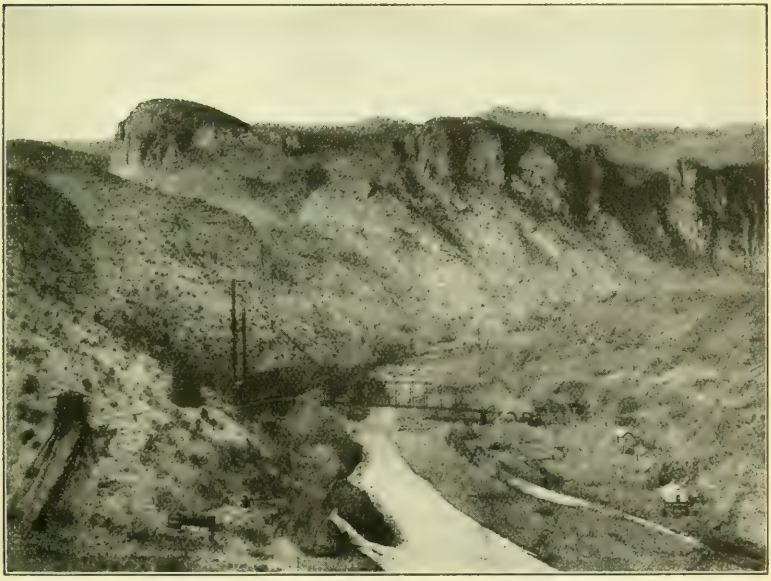


FIG. 4.—PUMPING PLANT, MORENCI WATER COMPANY.

structural steel frame on a concrete foundation, with corrugated-steel sides and roof. The addition is of corrugated-steel construction, but with wood frame. In the main engine-room, there is a 10-ton, Maris Brothers, hand, traveling crane of 45-ft. span. The floor of this room is of reinforced concrete, but that of the smaller engine-room is of wood. The basement floors of these and the boiler-room are on the same level and are of concrete. The building is well lighted.

Fig. 5 is a diagrammatic plan of the plant, showing the principal features.

A steel bridge, with a span of 188 ft. and a width of 9 ft., carries the three 10-in. water mains over the Eagle River.

There are three steel fuel-oil storage tanks at the plant. A 100 000-gal. tank, 30 ft. high, and a 40 000-gal. tank, 18 ft. high, are on the same side of the river as the plant, and a 200 000-gal. tank, 30 ft. high, is on the opposite side.

The settling basins are along the cliff above the plant. Their location relative to the plant is shown on Fig. 16. They are described in more detail later.

On the side of the river opposite the plant, there are six cottages for the employees of the company. They are of substantial wooden construction, each having electric lights, running water, etc.

Equipment.

In the main engine-room there are two Nordberg, horizontal, four-cylinder, triple-expansion, condensing, poppet-valve, direct-acting, crank and fly-wheel type, pumping engines. The smaller engine-room contains one Nordberg, horizontal, cross-compound, condensing, direct-acting, crank and fly-wheel type, pumping engine, having poppet-valves on the high-pressure and Corliss valves on the low-pressure cylinders. The pump end of this engine extends into the main engine-room in order to be within reach of the 10-ton traveling crane. In both the triple-expansion and the cross-compound pumping engines, the cranks are set at an angle of 90 degrees.

Triple-Expansion Pumping Engines.—The arrangement of the triple-expansion, poppet-valve, pumping engines is similar to that of an engine at the Gneisenau shaft of the Harpener Bergwerks Gesellschaft, in Germany. The pump end in each engine is behind the steam cylinders, of which there are four. There are no connections

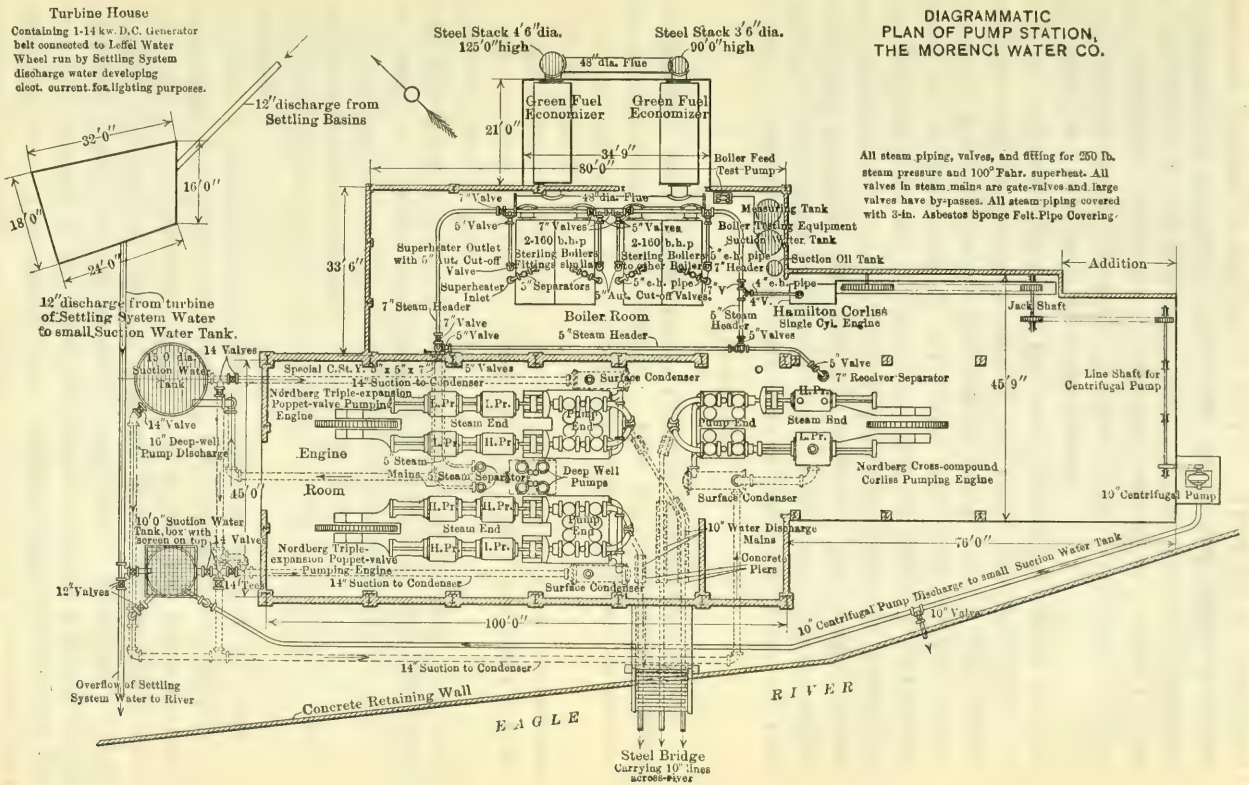

DIAGRAMMATIC
PLAN OF PUMP STATION,
THE MORENCI WATER CO.
Turbine House
Containing 1-14 kw. D.C. Generator belt connected to Leffel Water Wheel run by Settling System discharge water developing elect. current for lighting purposes.
12" discharge from Settling Basins
32'0"
18'0"
16'0"
24'0"
12" discharge from turbine of Settling System Water to small Suction Water Tank.
Steel Stack 4'6" dia. 125'0" high
Steel Stack 3'6" dia. 90'0" high
48" dia. Flue
Green Fuel Economizer
Green Fuel Economizer
21'0"
34'9"
80'0"
Boiler Feed Test Pump
All steam piping, valves, and fitting for 250 lb. steam pressure and 100° Fahr. superheat. All valves in steam mains are gate-valves and large valves have by-passes. All steam piping covered with 3-in. Asbestos Sponge Felt Pipe Covering.
7" Valve
48" dia. Flue
5" Valve
7" Valves
2-160 b.h.p Sterling Boilers
5" Valves
2-160 b.h.p Sterling Boilers
Measuring Tank
Boiler Tubing Equipment
Suction Water Tank
Superheater Outlet with 5" Aut. Cut-off Valve
Superheater Inlet
5" Separators
5" e.h. pipe
5" Aut. Cut-off Valves
5" e.h. pipe
7" Header
Suction Oil Tank
4" e.h. pipe
Addition
33'6"
7" Steam Header
7" Valve
5" Valve
Boiler Room
5" Steam Header
5" Steam Header
Hamilton Corliss Single Cyl. Engine
5" Valves
Jack Shaft
45'0" Suction Water Tank
14" Valves
Special C.S.L.Y. 9" x 5" x 7"
5" Valves
Surface Condenser
5" Valve
7" Receiver Separator
45'9"
Line Shaft for Centrifugal Pump
14" Suction to Condenser
Nordberg Triple-expansion Poppet-valve Pumping Engine
L.Pr.
Pump End
H.Pr.
Steam End
14" Valve
16" Deep-well Pump Discharge
Engine
5 Steam Mains
Steam Separator
Deep Well Pumps
Steam End
Pump End
Steam End
L.Pr.
Nordberg Cross-compound Corliss Pumping Engine
45'0"
Room
10'0" Suction Water Tank, box with screen on top
14" Valves
H.Pr.
L.Pr.
Surface Condenser
10" Centrifugal Pump
10" Water Discharge Mains
Concrete Piers
76'0"
12" Valves
14" Tees
Nordberg Triple-expansion Poppet-valve Pumping Engine
14" Suction to Condenser
Surface Condenser
100'0"
10" Centrifugal Pump Discharge to small Suction Water Tank
10" Valve
14" Suction to Condenser
Overflow of Settling System Water to River
Concrete Retaining Wall
EAGLE
RIVER
Steel Bridge
Carrying 10" lines across River

FIG. 5.

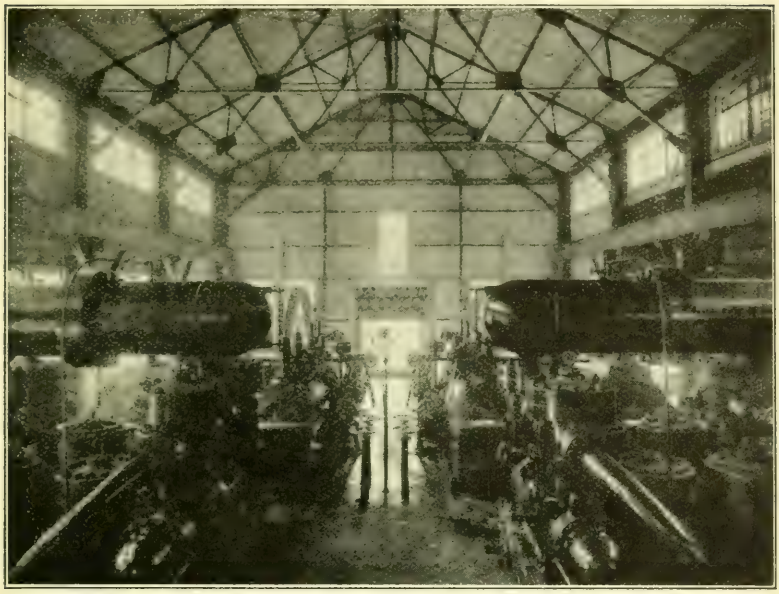


FIG. 6.—TWO NORDBERG, TRIPLE-EXPANSION PUMPING ENGINES.

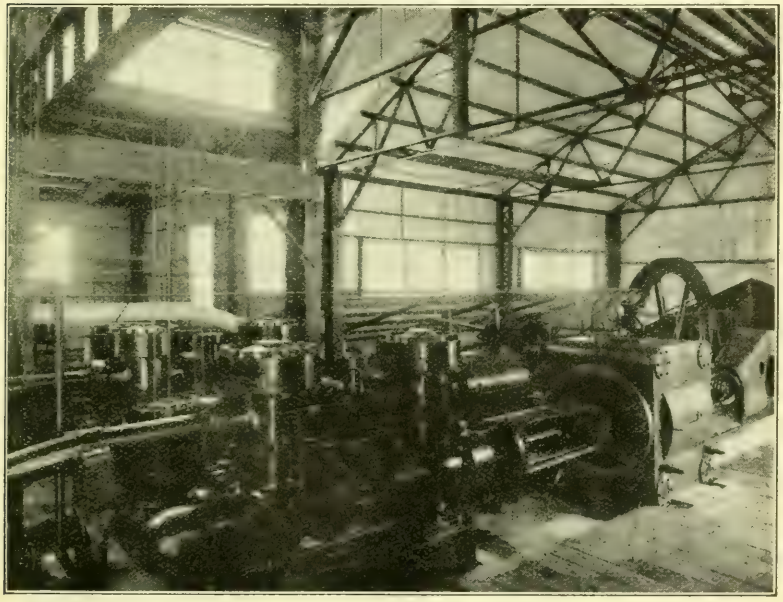


FIG. 7.—NORDBERG CROSS-COMPOUND PUMPING ENGINE.

between the steam end and the pump except the bed-plate, which is sufficiently strong to absorb all the strains between the pump and the steam cylinders. The steam end of each engine consists of a high-pressure cylinder and an intermediate-pressure cylinder, each of which is in tandem with and attached by a strong distance piece to one of the low-pressure cylinders, of which there are two. The intermediate- and low-pressure cylinders are steam-jacketed—in both the barrel and the heads—but the high-pressure cylinder is not jacketed. Each cylinder has four poppet-valves. They are operated by a lay shaft, along each side of each engine, which is driven from the main shaft. The high-pressure cylinder has a variable cut-off under control of the governor. The cut-off on the intermediate- and low-pressure cylinders is fixed. Each engine has two receivers of the re-heater type. The receivers, main throttle valve, steam and exhaust mains, other piping, and all auxiliaries are under the floor of the engine-room. All the auxiliaries, namely, the boiler feed-pump, oil-separator drain-pump, jacket drain-pump, and air-pump for each pumping engine, are of the attached type, being driven from the cross-head, on the intermediate-cylinder side between the steam and pump ends of the main pumping engines, through a rocker shaft and arms. From the cross-head on the high-pressure cylinder side of each pumping engine a set of deep-well pumps is driven. There is a cylindrical surface condenser with 1 000 sq. ft. of cooling surface for each pumping engine. The exhaust steam main to each condenser contains a Cochrane oil separator.

The deep-well pumps lift the water for the main pumps, from a well about 60 ft. deep, and deliver it into the large suction water tank from which it flows by gravity to the suction chambers of the main pumps. The condenser is in the suction to the main pumps, and the water on its way to the pump suction chambers makes a single pass through the condenser tubes. The steam to be condensed comes in contact with the outside of the condenser tubes. The condensate from the condenser flows by gravity to the air-pump. The latter discharges the condensate into a small steel tank hot-well from which the boiler feed-pump delivers it to economizers. The make-up water for the boiler feed is admitted into the condenser at the exhaust steam inlet.

The pump end is of cast iron, and was designed especially strong to work against the heavy total hydraulic head of about 1 700 ft. The pumps are of the horizontal, duplex, double-acting, outside, center-packed plunger type. The plungers are of chilled cast iron. The pump-valves are of the double concentric ring type, of cast steel, faced with fiber washers. The seats are of cast iron, strongly ribbed, and have a central stud of brass passing through and guiding the valve. Each valve seat has an open area of 65 sq. in. The lift of the valve can be controlled by the adjustment of a spring. The total weight of a pump-valve and seat is about 425 lb. Each pump contains eight valves—four suction and four discharge—one valve to a chamber, as shown on Fig. 8.

Each triple-expansion pumping engine is designed to pump "1 000 U. S. standard gal. of water per min., against a total head of 750 lb., at a piston speed of 300 ft. per min.; and, when running condensing, is guaranteed to do 175 000 000 ft-lb. of work for every 1 000 lb. of steam evaporated from hot-well temperature with a steam pressure at the throttle of 165 lb., gauge, and the steam superheated 100° Fahr."

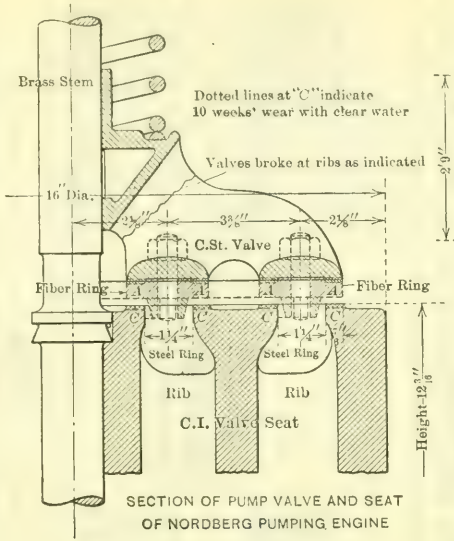
Table 1 gives the results of a duty test on one of these pumping engines by a responsible testing engineer. The quantity of steam to be charged to the engine was obtained by weighing the steam condensed by the condenser and in the jackets and re-heaters. The leading dimensions of this engine are given in Table 2.

TABLE 1.—Test of Triple-Expansion Pumping Engines.

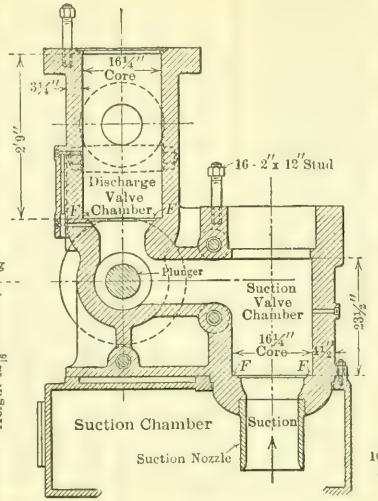
Duration of test	8 hours.
Average steam pressure at throttle	170 lb., gauge.
Average steam pressure at first receiver	34 "
Average steam pressure at second receiver	0 "
Average superheat at throttle	142° Fahr.
Average water pressure	720 lb.
All water pumped was also lifted out of a well a distance of	35 ft.
Duty per 1 000 lb. of steam	181 197 500 ft-lb.
Steam per indicated horse-power per hour	10.23 lb.
Mechanical efficiency	93 per cent.

The pumping engine ran slightly over capacity.

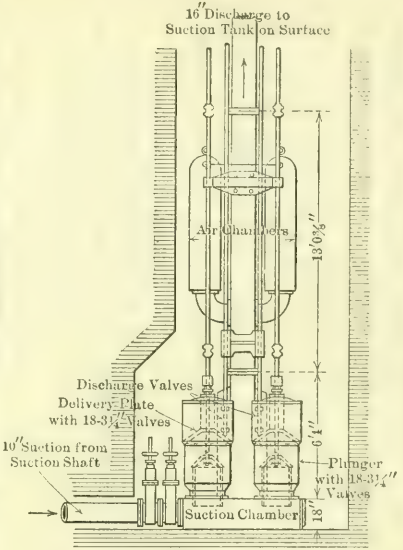
Cross-Compound Pumping Engine.—The horizontal, cross-compound, pumping engine was designed to pump "1 000 U. S. standard gal. of water per min., against a total head of 750 lb., at a piston speed of 300 ft. per min.; and when running condensing, guaranteed to do 160 000 000 ft-lb. of work for every 1 000 lb. of steam evaporated



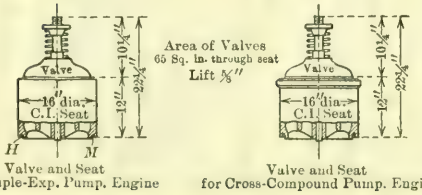
SECTION OF PUMP VALVE AND SEAT OF NORDBERG PUMPING ENGINE



SECTION OF PUMP VALVE CHAMBERS



GENERAL ARRANGEMENT OF DEEP-WELL LIFT PUMPS AT BOTTOM OF DRY WELL



Valve and Seat for Triple-Exp. Pump. Engine

Valve and Seat for Cross-Compound Pump. Engine

VALVE CHAMBERS, VALVES, SEATS AND DEEP-WELL LIFT PUMPS, THE MORENCI WATER CO.

FIG. 8.

TABLE 2.—Principal Dimensions of Triple-Expansion Pumping Engines.

High-pressure steam cylinder	20-in. diameter, 42-in. stroke.
Intermediate-pressure steam cylinders	38-in. diameter, 42-in. stroke.
Low-pressure steam cylinder	40-in. diameter, 42-in. stroke.
Plungers (two)	6¾-in. diameter, 42-in. stroke.
Plunger rods (two)	2 13/16-in. diameter.
Piston rods, head end, high-pressure and intermediate-pressure cylinders	3⅞-in. diameter.
Piston rods, head end, low-pressure cylinders and crank end, high-pressure and intermediate-pressure cylinders	4 11/16-in. diameter.
Piston rods, crank end, low-pressure cylinders	4⅞-in. diameter.
Fly-wheel (weight = 27 000 lb.)	16-ft. diameter, 10 in. wide.
Main bearings	13¾ by 25 in.
Crank pins	6¾-in. diameter, 15 in. long.
Cross-head pins	5-in. diameter, 8½ in. long.
Cross-head shoes	12 by 20 in.
Foundation dimensions	14 ft. 0 in. by 57 ft. 1⅛ in.

from hot-well temperature with a steam pressure of 155 lb., gauge, at the throttle, and the steam superheated 100° Fahr."

Table 3 gives the results of a duty test on this engine, running slightly over capacity, by a responsible testing engineer. The quantity of steam used was determined as in the case of the test on one of the triples.

TABLE 3.—Test of Cross-Compound Pumping Engine.

Duration of test	10 hours.
Average steam pressure at throttle	155.7 lb., gauge.
Average water pressure	725.5 lb.
Average superheat in steam at throttle	107.8° Fahr.
Mechanical efficiency	96 per cent.
Duty per 1 000 lb. of steam	171 620 000 ft-lb.
Steam per indicated horse-power per hour	11.1 lb.

The pump end and pump valves of this engine, in the main, are identical with those of the triple-expansion engines. There are two steam cylinders—a high-pressure and a low-pressure. The high-pressure cylinder has four poppet-valves, and these are operated by a lay shaft, along the side of the engine, driven from the main shaft. The cut-off is variable, being under the control of the governor. The low-pressure cylinder has Corliss valves, with the cut-off controlled by hand. The high- and low-pressure cylinders are both jacketed; the former in the heads only, the latter in both barrel and heads. The auxiliaries are of the attached type, being driven from the cross-head between the pump and steam ends, as in the triple-expansion engines. This pump, however, has no deep-well lift-pumps attached,

as it pumps principally settling system water for the concentrators. The condenser and air-pump combined are similar to those of the triple-expansion engines.

The leading dimensions of this pumping engine are given in Table 4.

TABLE 4.—Principal Dimensions of Cross-Compound Pumping Engine.

High-pressure steam cylinder	23-in. diameter, 42-in. stroke.
Low-pressure steam cylinder	52-in. diameter, 42-in. stroke.
Plungers (two)	6¾-in. diameter, 42-in. stroke.
Plunger rods (two)	2 13/16-in. diameter.
Main bearings	13¾ by 25 in.
Crank pins	6¾-in. diameter, 15 in. long.
Cross-head pins	5-in. diameter, 8½ in. long.
Cross-head shoes	12 by 20 in.
Piston rods, crank ends	4⅞-in. diameter.
Piston rods, head end	3⅞-in. diameter.
Fly-wheel (weight = 27 000 lb.)	16-ft. diameter, 10 in. wide.
Foundation dimensions	18 ft. 0 in. by 47 ft. 10 in.

The triple-expansion and cross-compound pumping engines are equipped with Richardson force feed oil pumps for steam cylinder lubrication; and for the lubrication of the bearings, etc., there is a complete gravity oiling system, with filters, oil pumps, tanks, etc. Wherever practicable, grease cups are used.

Operating Experiences.

Usually, it is necessary to run two pumping engines, and at such rates as the demand for water requires, for the water storage at Morenci is very limited. Consequently, the rate of running is not regular and not as uniform as is usually the case in the average water-works pumping plant. The average duty developed by these pumps, under the usual operating conditions, during a period of one month, varies from 155 000 000 to 162 500 000 ft-lb. per 1 000 lb. of steam chargeable to the engines; that is, the duty developed comes within from 10 to 5% of that which would probably be developed under similar test conditions. Experience has shown that the plant performance is not as good when two triples are running as when a triple and the cross-compound are running. Under the usual conditions, the cross-compound produces better results back at the boiler end than a triple, although, under test conditions, the results favor the triple.

As is perhaps usual, to a certain extent, in machinery of new and special design, unsatisfactory features developed in these pumping engines which led to a great deal of trouble and expense. The pump valves caused the most serious trouble. The original pump valves were of cast steel, and the seat was of cast iron, weighing approximately 400 lb. These valves were not suitable for the conditions and proved extremely unsatisfactory, as they generally began to leak within a few days after being placed in the pump. Under favorable conditions, with clear water, the wear on valve and seat, in the course of 10 weeks, was as indicated by the dotted lines at *A* and *C* on Fig. 8. Consequently, both valve and seat had to be refaced at considerable expense once every 10 weeks and sometimes oftener. In addition to this heavy repair expense, the life of the valve seat was naturally very short. The wear on the seat was usually uneven, and caused the ribs of the valve to crack, as indicated on Fig. 8. The total maintenance cost of this item of valves and seats was extremely high. Furthermore, with the use of these valves, the quantity of water actually pumped was from 72 to 85% of the plunger displacement. Finally, a cast-steel valve, faced with a fiber washer, was tried. This is shown by the light dotted lines on Fig. 8. These washers are ½-in. thick, are held in place by ½-in. studs and a steel ring, are able to withstand the great pressure to which their bearing surface is subjected, and are giving satisfaction. The wear has been transferred from the 400-lb. cast-iron seat to the fiber washers, which are easily replaced. These washers leak a trifle when a valve is first placed in the pump, but soon pound themselves to the seat, after which they never leak. The life of a set of washers is about 4 months.

Another unsatisfactory feature developed in connection with the pump valve seats. Fig. 8 shows a section through part of a pump valve and seat, and a section of the pump chambers. When a valve and seat are placed in a chamber, the bottom of the valve seat, *H*, rests on the bottom of the chamber, *F*. A round gasket is placed at *M*. The result is a joint approaching a ground joint. A spindle fits into the top of the brass stem and is screwed down on it after the cover has been placed on the chamber. This is the only means, outside of the action of the water pressure, and its own weight, of forcing and holding the valve seat down on the chamber bottom in the case of the triples, and is not capable of insuring positively such a result.

It does not accomplish its purpose with absolute certainty in the very place where such a feature is of vital importance. As a consequence, with the original cast-steel valves, a leaky condition frequently existed at the bottom of the valve seats, and there was no means of detecting it. There was constantly the danger of the bottoms of the valve chambers becoming seriously cut.

In the case of the cross-compound pumping engine, which was erected later, the design was changed by putting a beveled shoulder on each valve seat, on which four lugs, in the side of each valve chamber, acting as wedges, are forced. This design has proved more satisfactory, but not entirely so. With the present pump valves faced with fiber, a leak at the bottom of the seat can usually be detected, and so the danger of seriously cutting the chamber bottoms is not so great.

With the present valves faced with fiber, and changes in the throat bushings of the plunger stuffing-boxes, the average quantity of water pumped during 1913 amounted to 95.02% and during 1914, 95.3% of the plunger displacement.

The triple-expansion engines were originally equipped with air chambers, but as it was impossible to retain sufficient air in them to be of benefit, the pumps have been running without them. At this great water pressure, the air was either forced through the pores in the cast iron, of which the air chambers are made, or absorbed very rapidly by the water. There was no economical method of replenishing it rapidly enough. At the same time, up to date, there have been no difficulties in running without air chambers—due perhaps to the irregularity of the 10-in. lines conveying the water to Morenci. Without doubt, the small air pockets along the lines are sufficiently numerous to absorb the usual shocks due to the pulsations of the strokes of the plungers.

The tests of the pumping engines mentioned previously were conducted after the changes had been made in the pump valves and plunger stuffing-boxes, and other improvements had been effected.

Another very unsatisfactory feature in connection with the pumping engines is the vacuum-producing equipment with which they were furnished. The vacuum obtained is comparatively poor. The equipment for each engine consists of a cylindrical surface condenser and an air-pump of the attached type. The surface condensers for both

the triples and the cross-compound have each 1 000 sq. ft. of cooling surface, which, according to the usual practice, should be ample to produce sufficient cooling to obtain a very good vacuum. Each condenser is 42 in. in diameter, and contains 270 1½-in. outside diameter 16 B. w. g., seamless drawn brass tubes, 9 ft. 6 in. long. The exhaust steam enters the condenser at one end at the top and comes in contact with the outside of the tubes. A baffle-plate extends along the upper portion of the condenser so that the steam makes one pass. The condenser is slightly inclined, and the condensate leaves it at the bottom, directly opposite the exhaust steam inlet, and flows by gravity to the air-pump. There are no baffles other than the one mentioned.

The type of air-pump furnished for each condenser is the single-acting, vertical, bucket, combined wet and dry type, 17 in. in diameter, with 16-in. stroke. In the case of the triples, the suction of the air-pump has no foot- or check-valve. The air-pump for the cross-compound has a foot-valve in the suction and a separate air connection with a valve from the surface condenser, which leads into the pump between the discharge-valve check and the bucket.

The altitude at the plant is about 3 600 ft. and the corresponding barometer reading about 26.5 in. Although, at times, there is a better vacuum, it generally averages 18.5 in. for the triples, the corresponding back pressure of which is 3.93 lb. This was the average vacuum from the day the pumps were first started. The temperature of the condensate, water vapor, and air mixture in the suction of the air-pump is 127° Fahr. On the cross-compound, the average vacuum is about 20.5 in.; the corresponding back pressure of which is 2.95 lb.; and the temperature of the condensate, water vapor, and air mixture in the suction of the air-pump is 135° Fahr. Consequently, in the case of the triples, the total of 3.93 lb. in the condenser is made up of 2.04 lb. steam pressure and 1.89 lb. air pressure; and, in the case of the cross-compound, the total of 2.95 lb. in the condenser is made up of 2.53 lb. steam pressure and 0.42 lb. air pressure. From these figures, it is quite apparent that there is insufficient cooling in both cases. The figures indicate, furthermore, that the air-pump displacement for the triples is not large enough to extract the mixture of condensate, air, and uncondensed vapor as completely as is necessary to maintain a good vacuum. The ratio of the air-pump displacement to the low-pressure cylinder displacement is 1:25 for the cross-com-

pound and 1:30 for the triples. For this type of attached air-pump, practice has established ratios of 1:9 and 1:16, which give continued good results. The quantity of circulating or cooling water is more than 100 lb. per pound of steam to be condensed, which is abundant. The rise in temperature of this water through the condenser is only about 10°, and the difference between the temperature of the condensate mixture and this water after it has passed through the condenser is between 50 and 70° Fahr. The insufficient cooling, therefore, is not due to a lack of sufficient cooling water to abstract the latent heat in the steam. In view of the fact that there is an abundance of water and that the extent of cooling surface is adequate, it would seem that the insufficient cooling is due to the fact that the design of the condenser is not the proper one to insure that all the surface is fully effective.

Although the question of vacuum is not mentioned in connection with the duty guaranties, the fact of the matter is that the design of the entire vacuum-producing equipment is not in line with the careful, high-class design of the rest of the steam end of the engines.

In general, considering the severity of the service, these engines have been running very smoothly, and the results have been very satisfactory.

The engines complete, with all auxiliaries, including condensers and air-pumps, were built especially for this service by the Nordberg Manufacturing Company, of Milwaukee, Wis.

Boiler-Room Equipment.

The boiler-room equipment consists of four Sterling, Class A, No. 11, water-tube boilers, set in two batteries, each battery containing two boilers. Each boiler is rated at 160 b.h.p., is constructed for a working steam pressure of 160 lb., is equipped with a Hammel oil-burning furnace, adapted to local conditions, and one Hammel improved oil burner. A fuel-oil pump delivers the oil to the burners where steam is used to atomize it. Each boiler is also equipped with the Foster attached type of superheater, guaranteed to superheat 6 000 lb. of steam per hour at 160 lb. steam pressure, 125° Fahr., with a drop in pressure through the superheater of 1½ lb. or less. On each boiler, in the tubing carrying the saturated steam, between the boiler nozzle and superheater inlet, there is a steam separator with its drain

connected to a trap. This separator extracts any undue quantity of moisture, and thus assists the superheater by delivering to it a comparatively dry saturated steam. It also protects the superheater in case a boiler foams and large quantities of water are carried over with the steam. The drain at the superheater inlet is also connected to a trap.

In the rear of each battery of boilers there is a Green fuel economizer, containing 120 tubes, with 1 440 sq. ft. of heating surface, completely encased in brick. The two economizers are protected by a building of corrugated steel, with a wood frame.

The boiler-room is the most important part of a power-plant, especially in such an isolated place as Morenci, and in a country where fuel is costly, as large losses may occur there. Too much care cannot be exercised to determine what changes to effect and the proper routine to establish in order to obtain the best results under the operating conditions.

A great deal of attention has been given to the boiler settings. After the brickwork had been placed in first-class shape and given several coats of paint, the outside of the setting, especially in the region of the furnace and combustion chamber, was covered with a coating of asbestos several inches thick. All points of air leakage were stopped up, and the ends of the steam drums and mud drum, the flues in rear of the boilers, economizer connections, and all small piping around the boilers were covered with 2 in. of asbestos. The entire settings and flues, etc., were then covered with several layers of thick, unbleached muslin and given several coats of thick paint. On the brickwork on the top of the boilers, a layer of asbestos several inches thick was placed, and this was covered with a layer of bricks and cement. All the main steam piping has been covered with J-M Asbesto-Sponge felted pipe covering, 3 in. thick. A daily inspection of all settings in use is made, and all points of air leakage are stopped up and painted. In this way, the possibility of air infiltration through brickwork and at connections is reduced to a minimum; and the settings are maintained in first-class condition continually. The proper quantity of baffling inside the boiler settings has been determined by experiment, and the furnaces have been built in a manner which brings the best results. When burning fuel oil, the construction of the furnace, together with the manner of bringing the air to the burners, is important.

Miscellaneous Reserve Equipment.

The principal reserve equipment consists of a Hamilton-Corliss valve, single-cylinder, 12 by 36-in. steam engine, which runs a 10-in. Krogh horizontal centrifugal pump through a jack-shaft and a line of shafting, as indicated on Fig. 5. This engine, in conjunction with the centrifugal pump, is held as a reserve unit to lift water from the river into the small suction water tank, in case of an accident to the settling system, or to one of the deep-well pumps. This engine also furnishes power for any other purposes.

Fuel Oil System.

At Morenci, the fuel oil is emptied directly from railroad tank cars into two steel receiving tanks. A Gould triplex plunger pump, with a capacity of 40 gal. per min., geared to a 10-h.p. Westinghouse, mill type, induction motor, for which electric power is purchased from one of the mining companies, pumps the oil through a 4-in. pipe to the top of a hill about 300 ft. high, from which it flows by gravity through about 26 000 ft. of 6-in. pipe to the plant. As this pipe line runs over very rough country, there are many inverted siphons in its length. It contains about 20 000 gal. of oil, the quantity varying somewhat with the season of the year. Whenever oil is pumped over at Morenci, it flows into a 200 000-gal. tank which is about 100 ft. above the other two tanks. The latter are then filled from the 200 000-gal. tank as occasion requires, but never from the tank out of which oil is being used. Oil is drawn into the two oil-measuring tanks, from either the 100 000-gal. or the 40 000-gal. storage tank. From the measuring tanks, the oil flows by gravity to the fuel-oil pump which heats it and delivers it to the burners under a pressure of 60 lb. This enables the firemen to control his fire better than would be the case if he depended on gravity flow, and the pump uses no more steam than would be required to heat the oil sufficiently to give a free flow to the burners under gravity feed. The total fuel oil regularly on hand at the plant amounts to 310 000 gal.

The fuel oil is from California, gravity 18° Baumé, at a temperature of 62° Fahr. Pumping at the rate of 36 gal. per min., through 1 110 ft. of 4-in. pipe, up the hill at Morenci, temperature of oil 70° Fahr., the pressure at the pump is 160 lb.; the static pressure is 140 lb. During warm weather, it requires about 3 hours to send

over to the plant 6 500 gal. of oil. During cold weather, however, it requires about a day. As a general thing, no difficulty is experienced during ordinary cold weather; but when it is extremely cold, it is necessary to build fires along the line at regular distances, to heat the oil, so as to enable it to be pumped.

Water Transmission Mains Between Plant and Morenci, and Water Storage Tanks at Morenci.

There are three 10-in. lines conveying the water from the plant to Morenci. Fig. 9 is a plan and profile of one of the lines showing also the wagon road to the plant. The lines are within view from this road for their entire distance. Each pump discharges through two 8-in. bends and a Y-connection into its own 10-in. line. There are no cross-over connections between the lines.

At Morenci, the lines discharge into two steel storage tanks, each 53 ft. 6 in. in diameter and 30 ft. high, and each having a capacity of approximately 500 000 gal. One is designated the Town Tank, and into it only well water is pumped. The town distributing system receives its water from the Town Tank, and in this way the people are supplied with well water for domestic purposes. The mining companies take their boiler feed water from the town system. The other tank is designated the Concentrator Tank, and into it is pumped the settling system water. It also receives the overflow from the Town Tank. The fluctuations in water level occur in the Concentrator Tank and, consequently, it is provided with a high- and low-water alarm attachment. From the Concentrator Tank, water is delivered to the concentrators of both mining companies through 10- and 12-in. pipe lines, and these two lines are entirely separate from the town distributing system. All the water pumped into the storage tanks is measured by Venturi meters as it leaves them.

The 10-in. pipe lines from the plant to Morenci are laid on the surface of the ground, and conform to the topography of the country over which they run. Each line is about 25 650 ft. long, and contains 3 000 ft. (near the plant), weighing 54.74 lb. per ft.; 18 000 ft., weighing 40 lb. per ft.; and 4 650 ft. (near Morenci), weighing 35 lb. per ft. The pipe near the plant is extra strong, $\frac{1}{2}$ in. thick. All pipe is of wrought steel, threaded for $2\frac{1}{2}$ in. with standard threads, and screwed together with recessed line couplings. Near the plant, each line has two

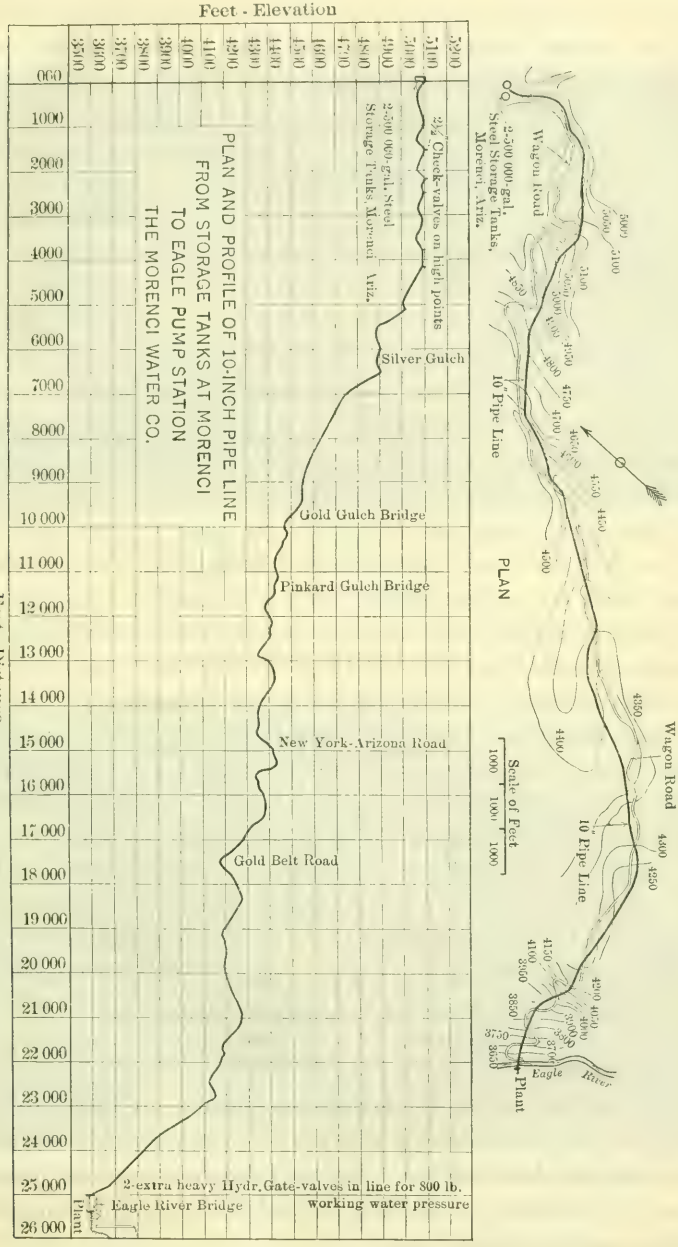

Feet - Elevation
Feet - Distance
PLAN AND PROFILE OF 10-INCH PIPE LINE
FROM STORAGE TANKS AT MORENCI
TO EAGLE PUMP STATION
THE MORENCI WATER CO.
2½ Check-valves on high points
2-500 000-gal. Steel Storage Tanks, Morenci, Ariz.
Silver Gulch
Gold Gulch Bridge
Pinkard Gulch Bridge
New York-Arizona Road
Gold Belt Road
2-extra heavy Hydr. Gate-valves in line for 800 lb. working water pressure
Eagle River Bridge
Plant
Wagon Road
10" Pipe Line
PLAN
Scale of Feet
Eagle River

Fig. 9.

10-in. Crane, flanged, extra heavy, hydraulic, ferro-steel, outside screw and yoke, wedge gate-valves, with hard metal seats and 1½-in. by-pass, designed for a cold-water working pressure of 800 lb.

Originally, each line contained also at this point a 10-in., Crane, extra heavy, hydraulic, ferro-steel, swing check-valve, designed for 800-lb. cold-water working pressure. Breaks in the lines between the pumping plant and the check-valves occurred with great frequency. It was noticed that the swinging disk of each check-valve pounded against its seat in unison with the pulsations of the pumps and set up a severe water-hammer. The check-valves were removed, and since then there have been no breaks in that portion of the lines.

There are no expansion joints in the pipe lines. The contraction and expansion in lines as exposed as these is very great, but is compensated by the large number of inverted siphons which they contain.

There are no fittings in the lines other than those mentioned. As the mains were being laid, all bends were made by heating the pipe in place and bending it to the desired deflection. Very satisfactory bends, which are neither flattened nor buckled, were made in this manner, and thus the lines were made to conform to the country with as few strains as possible in them. At the plant, the mains are carried over the Eagle River on a steel bridge and a wooden trestle.

Crane, extra heavy, hydraulic, ferro-steel, 10-in. flanges, suitable for a cold-water working pressure of 800 lb., are used in repairing breaks in the pipes. Each is 21 in. in diameter, 2¾ in. thick, and requires twenty 1¼-in. bolts. However, in the portion of the lines near the plant, these flanges were found to be unsatisfactory, as they cracked and gave way repeatedly. It was found necessary, therefore, to use a special flange, forged from a single billet of wrought steel, without weld, and finished to the same dimensions as the Crane ferro-steel flanges. Experience has shown that plain-faced flanges are best suited to the conditions, and that a ring gasket of vulcanized fiber, $\frac{1}{32}$ in. thick, is the only kind which gives complete satisfaction.

The lines are anchored only near the plant, where there are several large concrete anchor-blocks, well buried in the ground, and held in place with cast-iron saddles to prevent them from creeping down hill. Where the mains cross ravines and gulches, they are supported on wooden trestles, without floors.

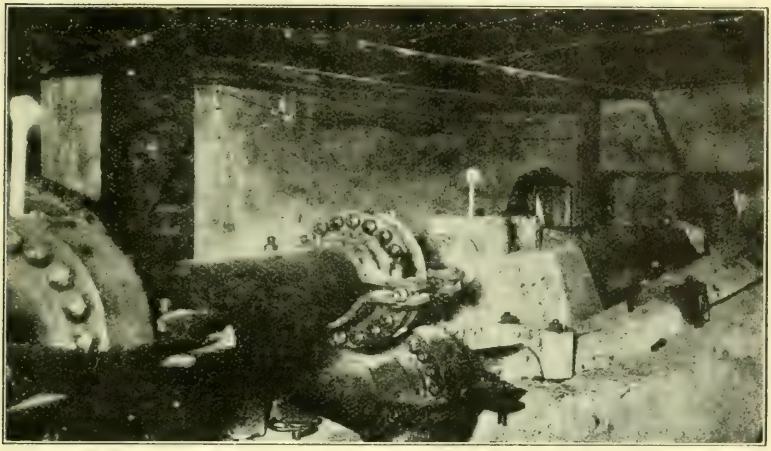


FIG. 10.—DISCHARGE PIPES FROM PUMPS IN PUMPING STATION, SHOWING METHOD OF ANCHORING.

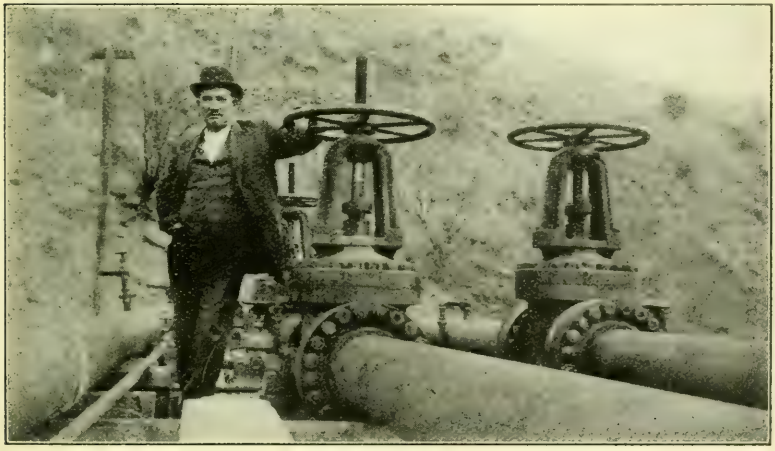


FIG. 11.—THE 10-INCH GATE-VALVES IN PUMPING MAINS NEAR THE PLANT.

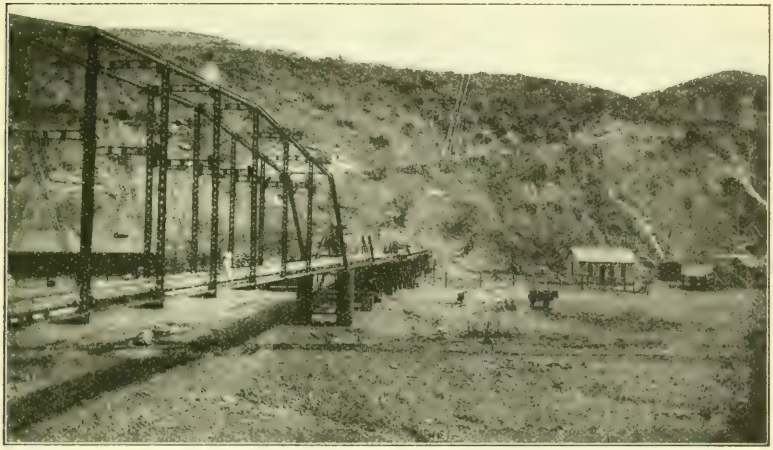


FIG. 12.—BRIDGE AND WOODEN TRESTLE CARRYING PUMPING MAINS ACROSS THE RIVER.

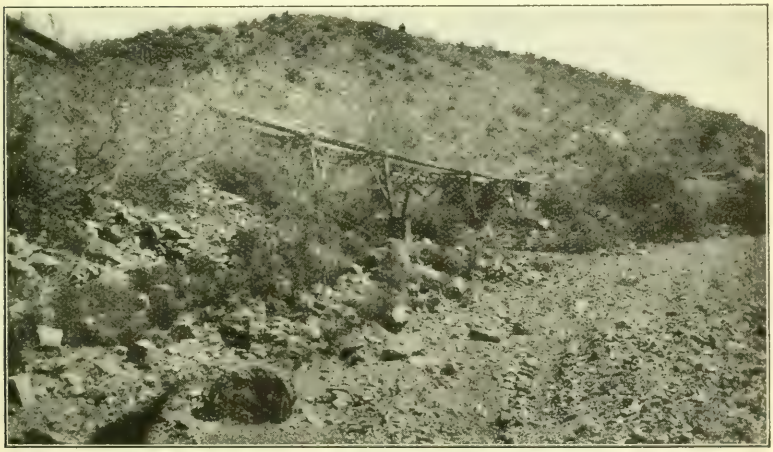


FIG. 13.—A VIEW OF THE PIPE LINES.

In the plant building, near the discharge of the pumps, it was found necessary to anchor each line thoroughly to piers and to support and brace all bends rigidly. At a point some 4 000 ft. from the water storage tanks at Morenci, the lines have very nearly the same elevation as the tops of the tanks. Here they form a siphon the vertical height of which between the extremities and the highest point is much more than 27 ft., which is about the theoretical height of a column of water that the air pressure at this altitude will sustain. Consequently, between pump pulsations, there is a break in the water column in each line at this point, and a partial vacuum forms, resulting in a very severe metallic water-hammer. Rapid corrosion of the pipe also takes place here. Thirty 2½-in., horizontal-swing, check-valves were placed on each line at this place, with a distance of 75 ft. between each. These admit air when there is a tendency for a vacuum to form, and confine the air when the pulsations force the water forward, thereby creating a cushion for any water-hammer. The air churns up with the water, resulting in an enlarged volume of a mixture of air and water. These valves have practically eliminated the water-hammer, and also, it is believed, the excessive corrosion of the pipes at this place.

The static head between the discharge of the pumps and the tops of the tanks at Morenci is 1 525 ft. When pumping through a 10-in. line, the total head amounts to 1 700 ft. This makes the friction head about 175 ft. in 25 650 ft. of smooth, screwed, wrought-steel pipe; or 6.82 ft. of friction head per 1 000 ft. of length.

Water-Supply Sources.

There are two sources of supply. Water for domestic and boiler feed purposes is obtained from a well. A settling system furnishes the large quantities used by the concentrators of the mining companies. Fig. 16 is a plan of this system, showing the tunnels, flumes, ditches, and settling basins, and their relative location with respect to the plant.

Wells.—The original well supplied all the water formerly pumped into Morenci, the maximum being about 300 gal. per min. This well consists of an unlined shaft, about 8 ft. square and 62 ft. deep, sunk in a conglomerate formation. A tunnel about 70 ft. long, also in conglomerate, connects this shaft with the gravel forming the river bed, at the edge of the conglomerate formation. This gravel bed is the source of the well water. With the building of the present plant, a

shaft 9 ft. 6 in. by 7 ft. 6 in. and 60 ft. 6 in. deep, containing the suction heads of the deep-well pumps, was sunk at a point some 80 ft. from the building. This shaft is in gravel, and is timbered for the first 22 ft. The remainder is in conglomerate. Two 10-in. suction pipe lines connect this shaft to the dry well, which is between the two Nordberg engines. The dry well is sunk to a depth of 62 ft. and is lined with concrete 3 ft. thick, with a clear 6 by 9-ft. opening. At the bottom of the dry well, there is an 8 by 12-ft. station containing the four cylinders of the deep-well pumps. Their construction is shown on Fig. 8. Each deep-well pump has two cylinders, and is driven from a triple-expansion pumping engine, as already explained. Each of the 10-in. suction lines from the suction shaft connects directly to a deep-well pump, which lifts sufficient water for one pumping engine. There are no check- or foot-valves in the suction lines of these deep-well pumps. A tunnel through conglomerate connects the suction shaft with the original well.

It required but a short time, pumping at a rate of from 1 200 to 1 300 gal. per min., which was the quantity desired at the completion of the present plant, to demonstrate that the yield of the original well was about 700 gal. per min. Holes were dug along the river bank, and from there it was noticed that the water-table of the gravel forming the river bed had become lowered. It was evident that the ground-water which accumulates in the gravel of the river bed was not gathering rapidly enough, through the natural percolation of the surface water, to maintain a sufficient supply in the well. Continued pumping seemed to show, also, that the permanent supply was a portion of a definite underground flow, the main body of which seemed to be at a greater depth, according to all indications. To maintain the supply of the well at the desired quantity, holes were dug in the gravel bank, on the same side of the stream as the plant, about 700 ft. up stream, and water from the river was turned into them. Each hole was about 20 ft. square and 14 ft. deep. There are no difficulties in obtaining sufficient well water in this way until the rainy season comes on. At that time, the floods bring down muddy water and silt, and the latter seals the river bottom and the sides and bottoms of the sump holes, thereby reducing the percolation of water to such an extent that the well supply drops to about 700 or 800 gal. per min. Until last year, the additional water required at that season was pumped into the small suction-water

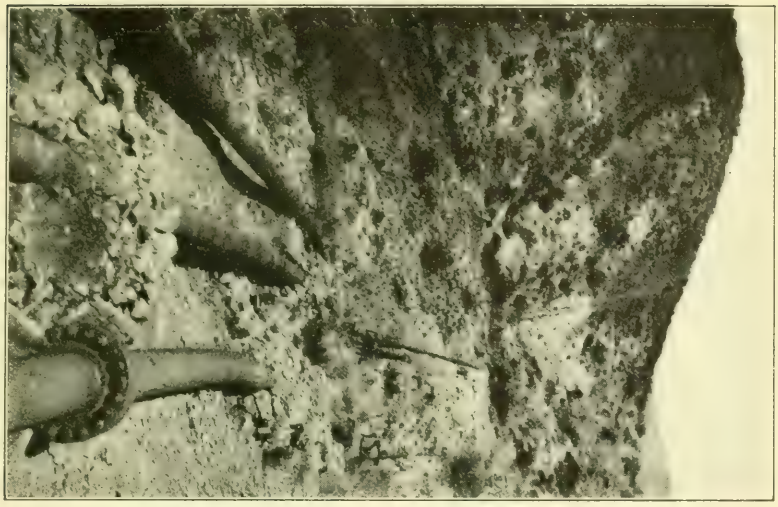


FIG. 14.—A VIEW ON THE PIPE LINES.

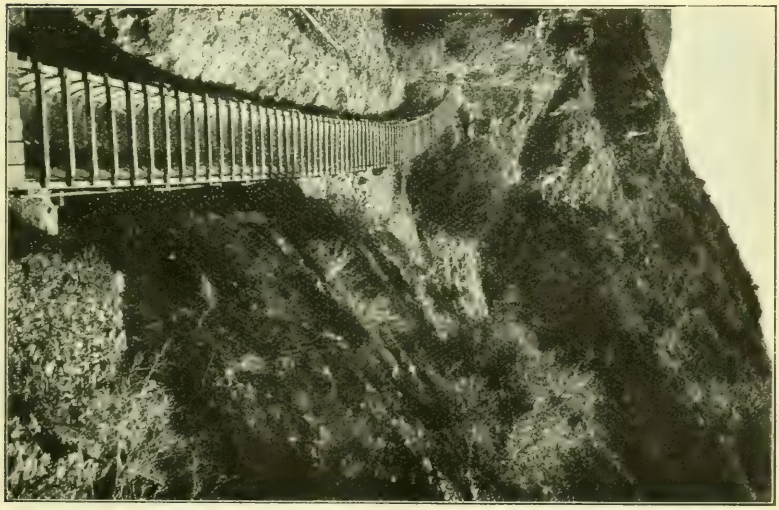


FIG. 15.—ONE OF THE FLUMES.

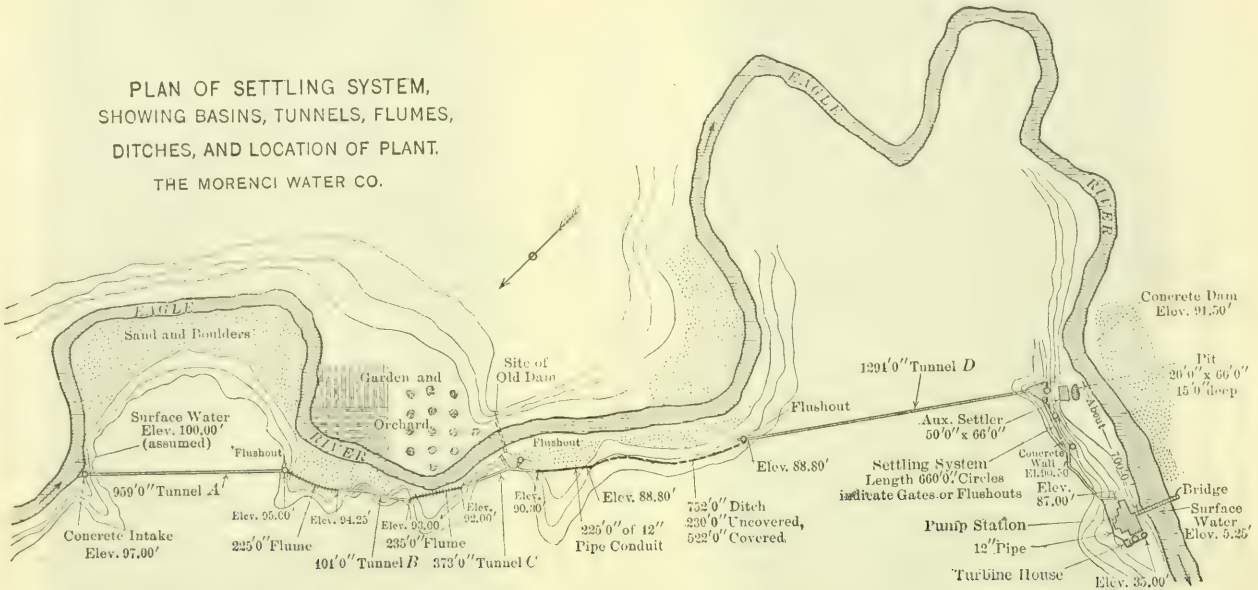

PLAN OF SETTLING SYSTEM,
SHOWING BASINS, TUNNELS, FLUMES,
DITCHES, AND LOCATION OF PLANT.
THE MORENCI WATER CO.
EAGLE
RIVER
Sand and Boulders
Garden and Orchard
Site of Old Dam
Surface Water Elev. 100.00′ (assumed)
Flushout
959′0″ Tunnel A
Concrete Intake Elev. 97.00′
Elev. 95.00′
225′0″ Flume
Elev. 94.25′
101′0″ Tunnel B
Elev. 93.00′
235′0″ Flume
Elev. 92.00′
373′0″ Tunnel C
Elev. 90.30′
Elev. 88.80′
225′0″ of 12″ Pipe Conduit
752′0″ Ditch
230′0″ Uncovered,
522′0″ Covered
Elev. 88.80′
1291′0″ Tunnel D
Aux. Settler 50′0″ x 66′0″
Settling System Length 660′0″ Circles indicate Gates or Flushouts
Concrete Wall El. 96.50
Elev. 87.00′
About 100.0
Concrete Dam Elev. 91.50′
Pit 20′0″ x 66′0″ 15′0″ deep
Bridge
Surface Water Elev. 5.25′
Pump Station
12″ Pipe
Turbine House
Elev. 35.00′

FIG. 16.

tank directly from the river by a centrifugal pump. From this tank, it was delivered to the concentrator-water tank in Morenci by a separate pumping engine and pipe line.

The pumping of muddy water during the rainy season very materially increased the pumping costs, seriously interfered with the operation of the plant, and presented many difficulties. The water contained so much silt and gritty matter that the condenser tubes were clogged to such an extent that the pumps could not be operated. It was necessary occasionally to clean the condensers several times in 24 hours. The grit in the water cut the pump valves and seats so much that they had to be changed every 3 or 4 days, and sometimes in less than 48 hours; also small sticks and foreign matter caused the valves to hang up, with the resultant injury to which pumps and pipe lines are subjected when under a total head of 1 700 ft.

At all times of the year, the well supplies sufficient water for domestic purposes and for boiler feed. It is delivered into the Town Tank by a separate pumping engine and pipe line. To obtain a sufficiently large quantity of satisfactory water for the increasing concentrator requirements, a settling system was installed.

Settling System.

The settling system consists of a series of settling basins, blasted out of the conglomerate along the cliff near, and some 65 ft. above, the plant. The first basin is formed by a concrete dam, about 14 ft. high, 5 ft. wide at the base, 2 ft. wide at the top, and 72 ft. long, built across a gulch at the outlet of Tunnel *D*, and has a water surface area of 3 600 sq. ft. This dam is well reinforced, resting on solid rock, and has an additional reinforcement of two abutments. A second settling basin, some 260 ft. long, 21.7 ft. average width, and containing a total water surface area of 5 600 sq. ft., was blasted out of solid conglomerate along the cliff. A concrete wall, 2 ft. wide and 3 or 4 ft. high, was built on the outer side of this basin, to make it of uniform elevation. In addition, there is the settling effect of the ditch, 300 ft. long and about 5 ft. wide, giving a water surface area of about 1 500 sq. ft. The total water surface area of the main settling basins is about 10 700 sq. ft. In addition to these main basins, there is an auxiliary settling basin, of concrete, 50 by 66 ft., which is beyond the reach of floods,

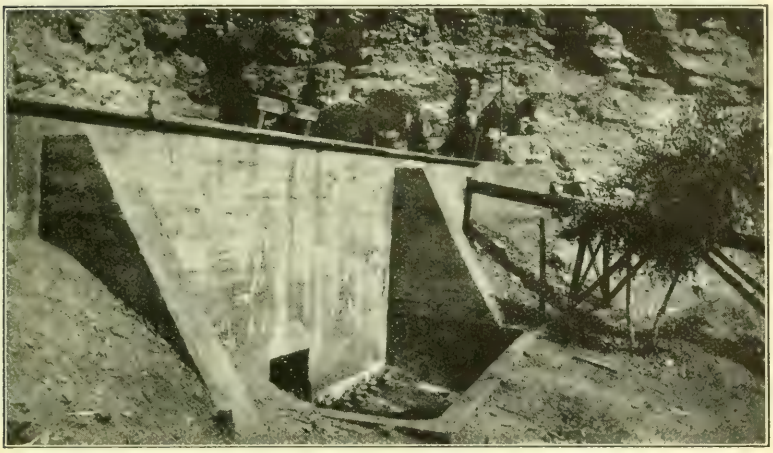


FIG. 17.—CONCRETE DAM ACROSS GULCH, FORMING FIRST SETTLING BASIN.

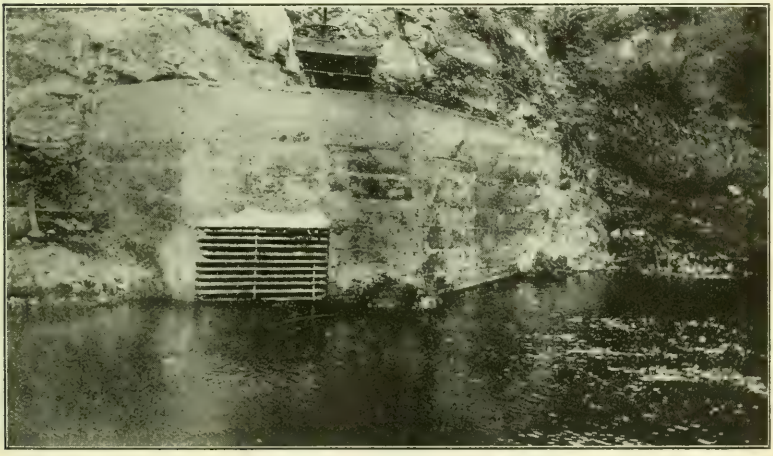


FIG. 18.—INTAKE OF SETTLING SYSTEM.

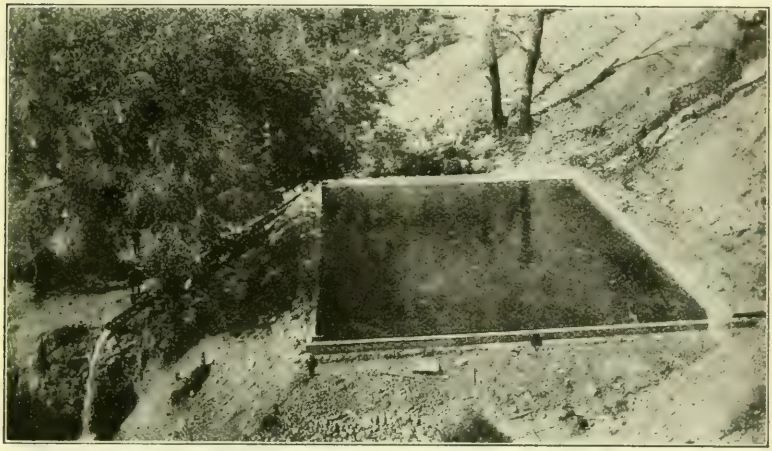


FIG. 19.—AUXILIARY SETTLING BASIN.

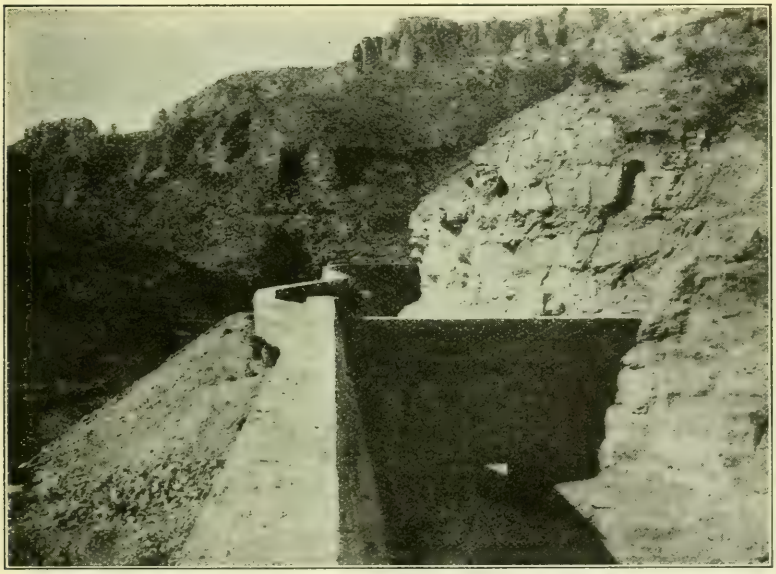


FIG. 20.—SECOND SETTLING BASIN, AND WALL BETWEEN FIRST AND SECOND BASINS.

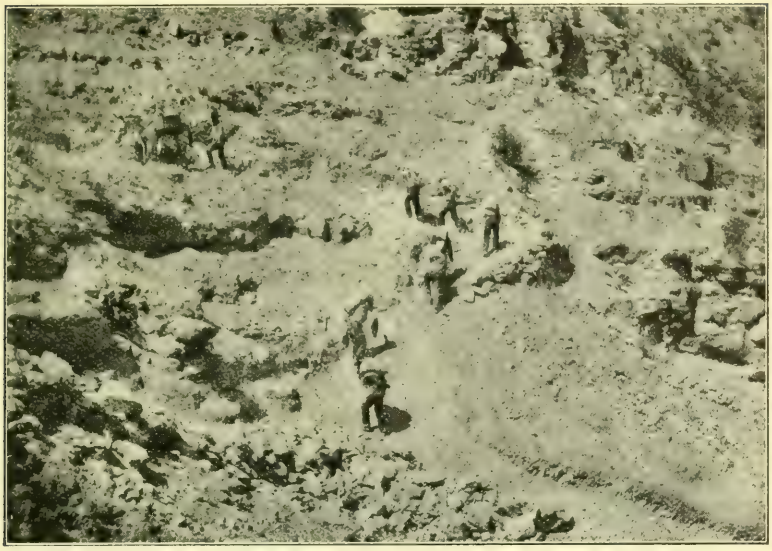


FIG. 21.—PACKING MATERIAL FOR WORK ON SETTLING BASINS.

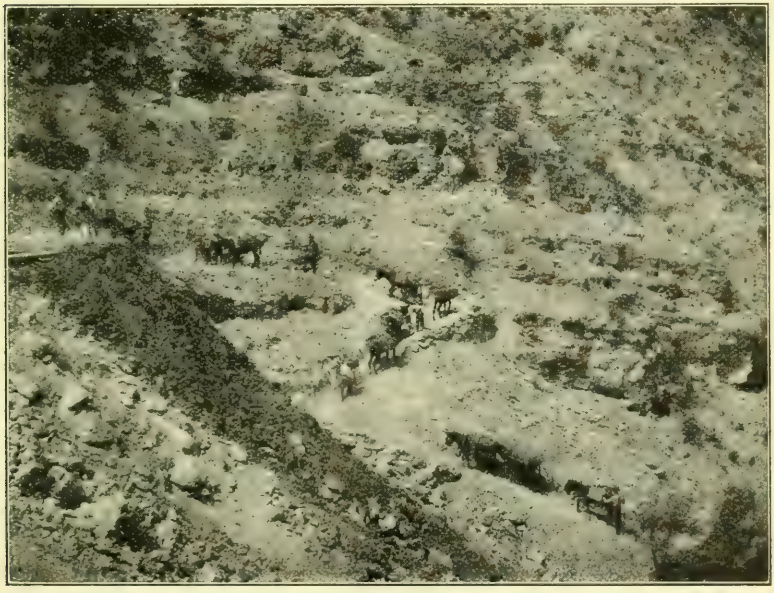


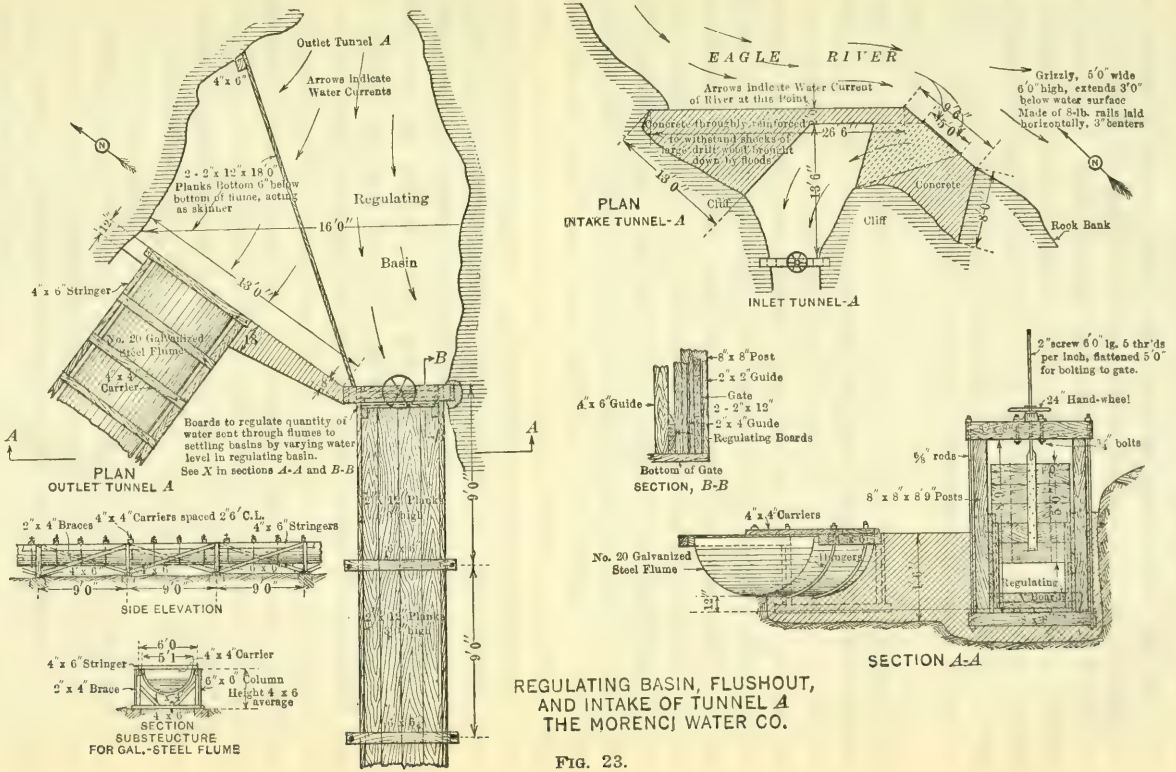
FIG. 22.—PACKING MATERIAL FOR WORK ON SETTLING BASINS.

along the river bank below and opposite the first basin. Water for it is taken from the first basin.

The water is brought to these basins from the river through 3 030 ft. of tunnels, 460 ft. of galvanized-steel semicircular flumes, about 5 ft. in diameter, 225 ft. of 42-in. pipe, acting as a conduit, and 752 ft. of ditch about 5 ft. wide.

The course of the Eagle River is very crooked, and runs through a canyon in the neighborhood of the plant. At a point up stream, about a mile in a straight line from the plant, the river makes a sharp turn, practically at right angles to its original direction of flow. The bank is of rock, almost perpendicular, and the intake of the settling system is at this point, as shown on Fig. 23. This intake is of concrete, well reinforced, and its top is 3 ft. above high water of usual floods. The position of the grizzly is such that the current during a flood has an opportunity to change its direction of flow completely, and so carry beyond it the greatest part of the driftwood, of which floods bring down very large quantities. The mixtures of concrete used were 1 part cement to 6 parts river gravel; and 1 part cement to 10 to 12 parts river gravel. The gravel found along the river consists of a mixture about in the proportion of 3 parts sand, and 5 parts rock, with some silt. The silt is less than 10%, if the location for obtaining the gravel is properly selected. All concrete work in connection with the settling system and around the plant consists of mixtures containing the river gravel just as found, and there have been no failures that can be traced to poor concrete because of the use of this gravel. In all concrete work, large stones have been worked in to as great an extent as possible. These were first wetted and then dropped into the concrete mixture already in place in the forms, and the mixture was then well worked around them.

At the outlet of Tunnel *A* there is a regulating basin, shown on Fig. 23. The water level in this basin is regulated by boards, *X*, and is maintained at an elevation at which sufficient water is sent through the galvanized-steel flume to the settling basins to supply the pumps, and there is a small quantity in excess. The excess water passes over the boards, *X*, through the wooden flume to the river, carrying with it the largest driftwood which passes through the grizzly. By removing the boards, *X*, the wooden flume serves as a flush-out for Tunnel *A*. Fig. 23 shows the leading dimensions and other information in con-



REGULATING BASIN, FLUSHOUT, AND INTAKE OF TUNNEL *A* THE MORENCI WATER CO.

FIG. 23.

nection with the design and construction of the steel flumes which carry the water over small gulches or washes.

All flumes, ditches, and tunnels are placed above high water of the greatest flood on record. The tunnels were driven either through rock or conglomerate, and are not timbered, with the exception of Tunnel *D*, which is partly timbered. The latter is driven through conglomerate for the entire distance, and at a number of places there are formations in the conglomerate, of a soft, talcy composition, which slakes and crumbles when air and water together reach it. These places have been timbered in order to prevent it from being choked up by the crumbling débris. All tunnels are 4 by 6 ft. The 42-in. pipe acts as a conduit through a wash which passes over the pipe. The pipe is built up of No. 10 iron, and came to the work in sections of 25 ft. One end is concreted into the end of Tunnel *C*; the other discharges into the ditch. The ditch is excavated on the side of a steep hill, and 522 ft. were covered in order to prevent it from being filled up by earth and rocks during heavy rainstorms. The covering was composed of timbers, the sets consisting of 6 by 6-in. posts at 6-ft. centers and 6 by 8-in. caps. The top only contains lagging consisting of 3 by 12-in. lumber. On top of this lagging there are several feet of earth. The covered portion of the ditch is 5 ft. wide and 6 ft. high. The average grade of tunnels, flumes, conduits, and ditches in the total length is 0.2 per cent. However, all the tunnels are constructed on steeper grades, and have flush-outs at the points indicated on Fig. 16.

Fig. 24 is a plan of the settling basins proper, with leading dimensions and information. Because of the great velocity of the river during floods, there is, in addition to suspended vegetable matter, silt, and float, a great deal of coarse, gritty sand which is kept suspended in the rapidly moving water. This system of basins was built to permit this gritty matter to settle, and incidentally as much of the silt and other matter as would deposit with it, by very materially reducing the velocity of the water. The water passing through the tunnels, flumes, and ditches has sufficient velocity to prevent practically all silt from settling on its way to the basins; yet the velocity is not great enough to cause any erosion of the sides of the ditch. At the outlet of Tunnel *D* the water flows into the first settling basin. The incoming water passes over the wall, *A*, in a sheet 28 ft. wide, and down the space between the wall and the baffle-boards to within a few

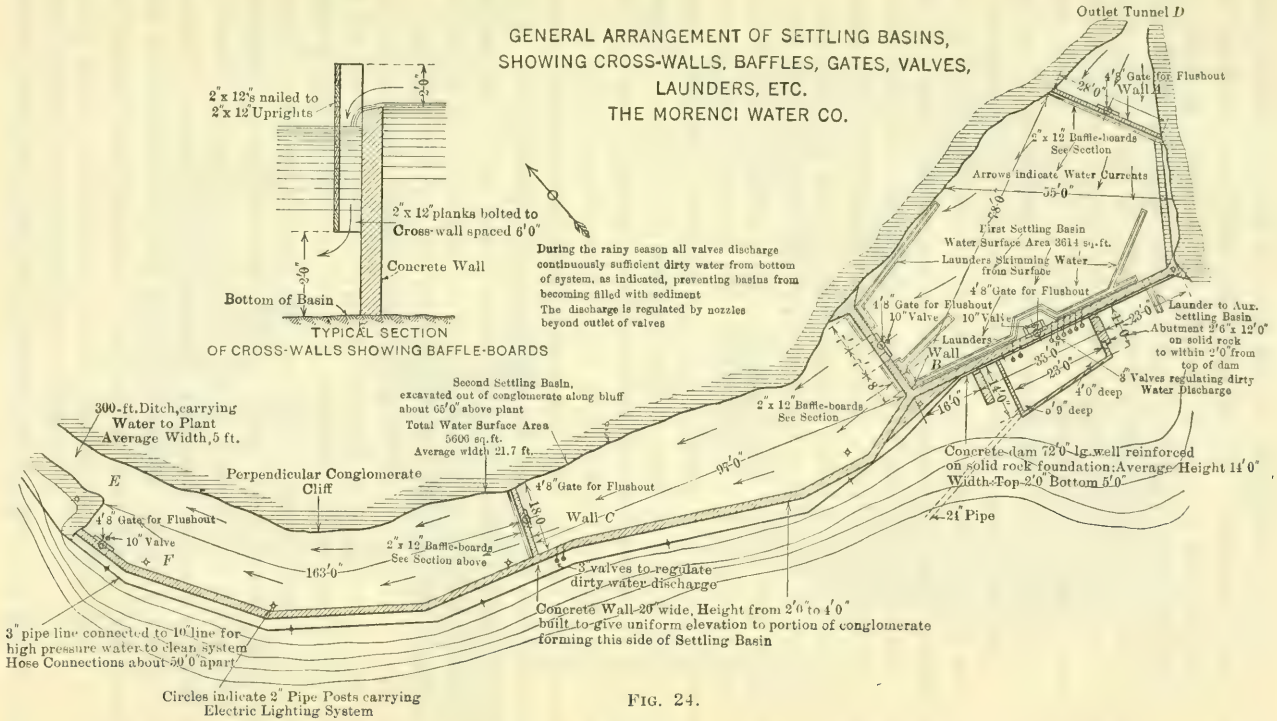

GENERAL ARRANGEMENT OF SETTLING BASINS,
SHOWING CROSS-WALLS, BAFFLES, GATES, VALVES,
LAUNDERS, ETC.
THE MORENCI WATER CO.
Outlet Tunnel D
4'8" Gate for Flushout
Wall A
2"x 12" Baffle-boards See Section
Arrows indicate Water Currents
55'0"
First Settling Basin
Water Surface Area 3614 sq.ft.
Launders Skimming Water from Surface
4'8" Gate for Flushout
10" Valve
Launders
Wall B
Launder to Aux. Settling Basin
Abutment 2'6"x 12'0" on solid rock to within 2'0" from top of dam
8 Valves regulating dirty Water Discharge
4'0" deep
0'0" deep
2"x 12" Baffle-boards See Section
Second Settling Basin, excavated out of conglomerate along bluff about 65'0" above plant
Total Water Surface Area 5606 sq.ft.
Average width 21.7 ft.
300-ft. Ditch, carrying Water to Plant
Average Width, 5 ft.
Perpendicular Conglomerate Cliff
E
4'8" Gate for Flushout
10" Valve
F
163'0"
2"x 12" Baffle-boards See Section above
Wall C
8 valves to regulate dirty water discharge
Concrete Wall 20' wide, Height from 2'0" to 4'0" built to give uniform elevation to portion of conglomerate forming this side of Settling Basin
Concrete dam 72'0" lg. well reinforced on solid rock foundation: Average Height 14'0" Width Top 2'0" Bottom 5'0"
21" Pipe
3" pipe line connected to 10" line for high pressure water to clean system
Hose Connections about 50'0" apart
Circles indicate 2" Pipe Posts carrying Electric Lighting System
2"x 12" nailed to 2"x 12" Uprights
2"x 12" planks bolted to Cross-wall spaced 6'0"
Concrete Wall
Bottom of Basin
TYPICAL SECTION
OF CROSS-WALLS SHOWING BAFFLE-BOARDS
During the rainy season all valves discharge continuously sufficient dirty water from bottom of system, as indicated, preventing basins from becoming filled with sediment
The discharge is regulated by nozzles beyond outlet of valves

FIG. 24.

feet of the bottom of the basin at a velocity reduced from that which it had in the tunnels and ditches. The water in its course then spreads out and flows at a very much reduced velocity to the surface, and the particles of gritty matter and silt settle to the bottom of the basins. Launders skim off the water in its course to the second settling basin and send it to the wall, *B*. There it is diverted to within a few feet of the bottom by baffle-boards, and continues to the top of the wall, *C*, at which point it is again diverted to the bottom of the basin. It flows from there toward *E*, where a ditch, about 5 ft. wide and 300 ft. long, carries it to a point near the plant. At the end of the 300-ft. ditch it flows through a 12-in. spiral riveted pipe to the small suction-water tank at the plant. This completes the course of the water through the settling basins from the outlet of Tunnel *D* to the point where the pumps receive it to deliver it into the concentrator tank at Morenci.

The settling basins are doing excellent work, eliminating all harmful sediment. The water is turbid in appearance, but there is practically no more trouble in pumping from this system during the rainy season than during the remainder of the year, when the water is clear. This system is intended to supply from 1 000 to 2 000 gal. per min.

Auxiliary Settling Basin.—At the point, *M*, in the first settling basin, about 500 gal. per min. are taken off and delivered by a launder to the auxiliary settling basin, and the water leaving it is sufficiently clarified so that it will not quickly seal the sides and bottom of the pit, 20 by 60 ft. and 14 ft. deep, excavated in the gravel bank beside this basin. The water from this pit percolates through the gravel of the river bed which forms a natural filter and during the rainy season maintains the supply from the well at about 1 000 gal. per min. This pit is about 700 ft. up stream from the plant, and repeated observations have demonstrated that the greater part of the water entering the pit seems to reach the well at the plant. It would hardly be practicable to increase the well supply sufficiently by this means, during the rainy season, in order to take care of the total water pumped, as the plant is in a canyon and a very limited area beyond the reach of floods is available. With fully 1 000 gal. per min. in the well during the rainy season, it is possible to run the well pump at its most efficient rate and have an abundance of well water for domestic use and boiler feed purposes.

Electric Lighting System at the Plant.

The water discharging from the settling system on its way to the small suction water tank, runs a Leffel water-wheel, to which is belted a General Electric, 14-kw., 125-volt, direct-current generator, at 1 425 rev. per min. Sufficient electric current is generated for lighting purposes at practically no cost. A small switch-board, with the necessary instruments, wiring, and lights, completes the equipment for furnishing light for the settling system, plant, bridge, employees' cottages, and buildings. The water-wheel and generator are protected by a corrugated-steel structure with a wood frame, well supported on a concrete foundation.

Construction.

A rough mountain wagon road, more than 6 miles long, constructed by the water company, extends from Morenci to the pumping plant on the Eagle River. A great part of the road is blasted along the rocky hillsides. Out of Morenci, the road runs up a hill requiring a climb 3 400 ft. in length, with grades varying from 17 to more than 36 per cent. Near the plant, there is a continual down grade along very rocky hillsides, with grades of 25 to 40 per cent. On steep grades, snubbing posts, were placed at regular intervals, during the construction period, for the purpose of letting the heavy loads down hill gradually. All material is hauled from Morenci to the plant on wagons. The average loads are about 3 000 lb. A load of this size requires ten horses to make the hill out of Morenci. During the construction period, the heavy pieces of machinery were taken out on a wagon built for heavy hauling, with 4-in. iron axles. Extra rear and front wheels were carried along to replace immediately any that broke down. Some of the pieces of machinery weighed from 15 000 to more than 24 000 lb. each. Twenty horses could make the hill out of camp with a load of 4 tons, and twenty-four horses could manage a load of 5 or 6 tons. It was necessary to use a number of triple blocks, with from six to eight horses pulling down hill, in order to work slowly a load heavier than 6 tons up the hill. At the plant, it was necessary to pack material and supplies around by Mexicans and burros, as in many cases the work was prosecuted at points inaccessible by any other method of transferring material. The cost of transferring machinery from the flat cars at Morenci to the plant, placed ready for the erector, averaged about $52.45 per ton, which is about twice

FIG. 25.—HAULING MATERIAL FROM MORENCI.

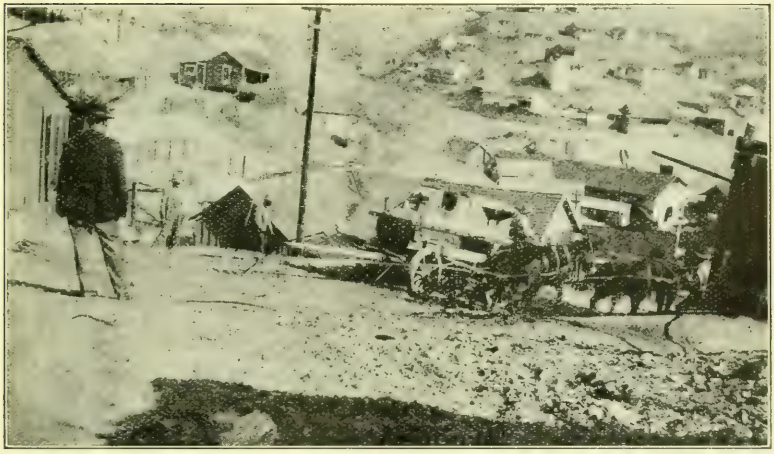


FIG. 26.—HAULING MATERIAL ON THE HILL OUT OF MORENCI.

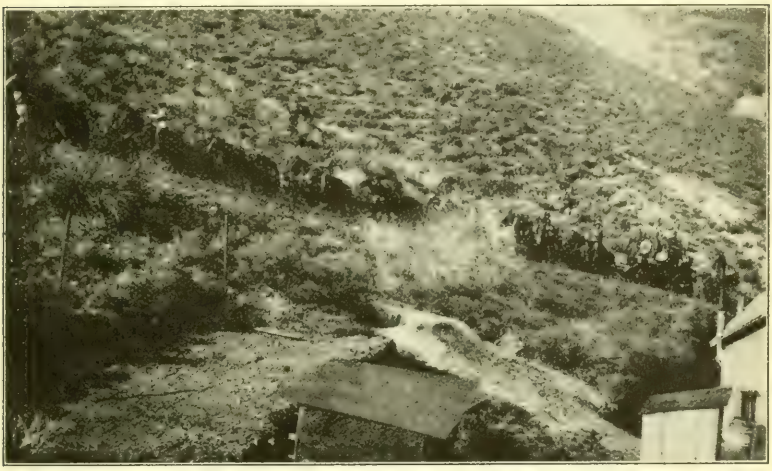


FIG. 27.—HAULING MATERIAL ON THE HILL OUT OF MORENCI.

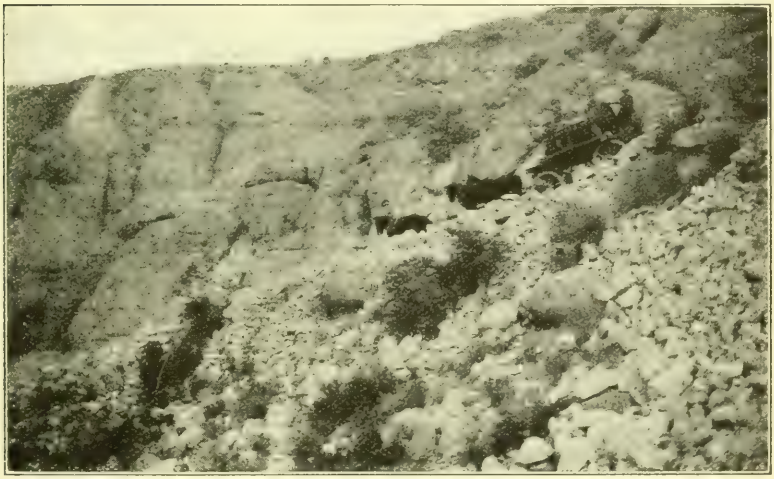


FIG. 28.—SNUBBING A LOAD DOWN A HILL, NEAR THE PUMPING STATION.

what the freight per ton amounted to from Milwaukee to Morenci. The 40-lb. and the 54.74-lb., 10-in. pipes were transferred through a tunnel and distributed by wagon to stock piles. In this way, the haul over the steep hill out of town was avoided. From the stock piles, the pipe was "snaked" by horses to the place where it was being laid. The tunnel was too small to permit passing heavy material through it. The cost of delivering the pipe from the cars to the location of the proposed line was $6.80 per ton. The average cost for each 10-in. pipe line laid complete, was $2.20 per ft. The average labor charge, for laying the lines by contract, was 28 cents per ft. for each 10-in. pipe line.

The cost of pumping engines complete, with foundations, auxiliaries, condenser, piping, etc., per 1 000 000 gal. capacity per 24 hours was:

Triple-expansion pumping engines	$38 500
Cross-compound " "	29 400
Average of all " "	35 500

The cost of the boiler plant, including piping, foundations, etc., was:

Without economizers—per rated boiler horse-power	$30.50
With economizers " " " "	42.50

The total rated capacity of the boiler plant is 640 b.h.p.

The cost of the pumping plant, including engines, boilers, economizers, piping, etc., per 1 000 000 gal. capacity per 24 hours was $41 500. These figures do not include anything for land, buildings, chimneys, wells, settling system, 10-in. pipe lines, etc., but practically only the items mentioned.

The capacity per pump per 24 hours is 1 500 000 gal., making the total 4 500 000 gal. Investigation will show that, considering the isolated location, the rough country, and the consequent difficulties of construction encountered, together with the high head to be pumped against, the total cost of the plant is reasonable.

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PAPERS AND DISCUSSIONS

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THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS

Discussion.*

By S. W. Bowen, M. Am. Soc. C. E.

Mr. Bowen.

S. W. Bowen,† M. Am. Soc. C. E. (by letter).—Referring to the use of "free" centers mentioned by Mr. Gayler, these were considered for the Fort Worth viaducts, but were not adopted for the following reasons: At West Seventh Street, there was not sufficient clearance between the high-water line and the underside of the arch rib to permit of their use. At Main Street, all the spans were of different lengths, which, at most, would have permitted the use of the centers only twice, once for each half of the structure. The ribs required considerable permanent reinforcement, so that in this case it was more economical to place the centers within the ribs, and use them both as centers and reinforcement.

"Free" centers have been used to great advantage in a number of large viaducts where there were several spans of the same length, and where little or no reinforcement was required in the arch, for example, for the "Tunkhannock", and other viaducts, built by the Delaware, Lackawanna, and Western Railroad Company. The first use made of these centers in the United States was for the Rocky River Bridge, at Cleveland, Ohio, mentioned by Mr. Gayler.

Pier 2, at West Seventh Street, is founded on a very dense mass of gravel and boulders. It was originally intended to place it

* Author's closure of the paper by S. W. Bowen, M. Am. Soc. C. E., continued from December, 1914, *Proceedings*.

† St. Louis, Mo.

Mr. Bowen.

on bed-rock, but, after an examination of the foundation, it was decided to stop at the gravel formation. The other piers were carried down to such a depth that, considering the ample bank protection, it seems highly improbable that there will be any trouble from scour. The reason for stopping the piles under Pier 2 at Main Street short of bed-rock is explained in the paper.

The arrangement for fastening the track rails did not prove to be as satisfactory as had been expected, principally on account of the difficulty of setting the anchor-bolts. This scheme was used in order to reduce the dead load on the structure.

In conclusion, the writer wishes to express his thanks to Mr. Gayler for his valuable discussion.

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SUBMERGED PIPE WORK AT PORTLAND, OREGON

Discussion.*

By Messrs. L. J. Le Conte and C. D. Ward.

Mr. Le Conte.

L. J. Le Conte,† M. Am. Soc. C. E. (by letter).—The writer is greatly pleased with this valuable paper. It is full of practical data based on actual experience, which, of course, makes it of particular value to the engineer in charge of water-works. There cannot be fair-minded discussion, however, unless all the local facts and conditions are known.

The work described is highly commendable in every respect, and the writer has only a few suggestions to make regarding ball-joints. He has had much sad experience with cast-iron ball-joints due to unexpected deflections in excess of the angular limit, 15° in this case. When the deflection is greater than this, the cast-iron ball-joint is likely to "snap", like a glass bottle. The writer, therefore, is firmly of the opinion that the less cast iron there is in a ball-joint the safer it is against extreme deflections. Accordingly, he presents a suggestion (Fig. 7) for a ball-joint which has much less cast iron and a greater deflection angle, 30° each way, and, in case there is an extreme jam or pinch, this ball-joint will stand the strain much better. A circular segment of a cast-iron shell is riveted to the extreme end of a pipe section. The socket into which this ball enters is in the form of a steel frustum of a cone, the length of which is about 1.5 times the diameter of the ball. The packing of the flange joints at the two ends may be made to conform to any of the standard methods based on experience. For the 30-in. lap-welded, steel pipe, this ball-

* Discussion of the paper by D. D. Clarke, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Berkeley, Cal.

Mr. Le Conte.

joint can be manufactured for $366, but the length of the joint is 7 ft., making the average cost $52.30 per ft. The cast-iron ball-joint described by Mr. Clarke costs $345, but the length is only 3 ft. 4 in., thus making the average cost $103.20 per lin. ft. Mounted on the plain siphon pipe, the average cost of these ball-joints per linear foot would be $19.50 for the author's siphon and $18.00 for

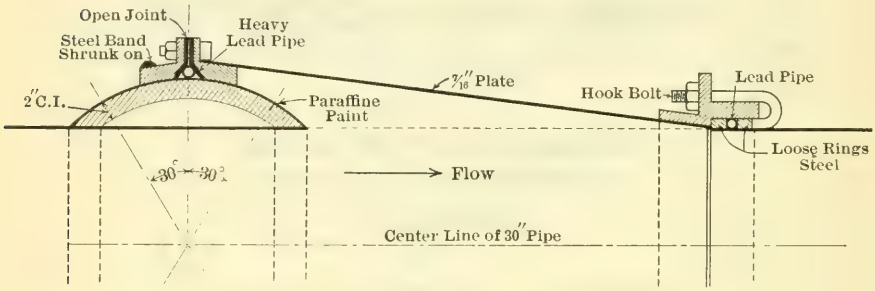

FIG. 7.

that of the writer. He believes, therefore, that it would have been quite possible to lay a better and safer siphon for a little less money.

Mr. Ward.

C. D. WARD,* M. AM. SOC. C. E.—Mr. Wiggin has mentioned the difficulty of moving a joint made with lead wool. As the speaker is not familiar with the use of this material, he would like to ask how hard the joint was caulked, and, if it had not been caulked so hard, would it have been possible to move it?

* New York City.

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PROOF OF AN ASSUMPTION IN THE THEORY OF CONCRETE BEAMS

Discussion.*

By J. P. J. Williams, Assoc. M. Am. Soc. C. E.

Mr. Williams.

J. P. J. Williams,† Assoc. M. Am. Soc. C. E. (by letter).—This mathematical proof of the validity of the usual assumption in reinforced concrete beam analysis, that plane sections remain plane after flexure, that is, that the steel strain is just sufficient to make this assumption correct, is very interesting. The almost universal respect accorded the method of least work should insure that this new application will strengthen materially the belief in the validity of the assumption discussed. It should be emphasized that this proof is also based on the assumption that the compression side remains plane, that is, the usual straight line variation for compressive stress is assumed. Hence the proof really applies to the tensile steel, assuming no tension in concrete, and shows that the work done by the steel to produce a minimum total work is that resulting from a strain just sufficient to keep the normal section a plane.

The writer finds that the analysis for T-beams can be made without much difficulty, if the variable be made z instead of k. In fact, this method of analysis is also much shorter for the case of the rectangular beam treated by the author.

Both these analyses follow, using the author's notation.

Rectangular Beam.

Write the equation for work, W_d, in terms of z, and finally solve for $k = \frac{3z}{d}$.

* This discussion (of the paper by Ralph E. Goodwin, Jun. Am. Soc. C. E., published in November, 1914, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Williams.

$$W_{dl} = \frac{1}{2}\ C \cdot \frac{2}{3}\ \frac{f_c}{E_c}\ dl + \frac{1}{2}\ T\ \frac{f_s}{E_s}\ dl.$$

Substitute $f_c = \dfrac{2\ C}{b\ kd} = \dfrac{2\ C}{3\ b\ z}$; $f_s = \dfrac{T}{p\ bd}$; $E_s = n\ E_c$

$$W_{dl} = \frac{dl}{E_c\ b}\left[\frac{2\ C^2}{9\ z} + \frac{T^2}{2\ pn\ d}\right].$$

Substitute $C = T = \dfrac{M}{d - z}$

$$W_{dl} = \frac{M^2\ dl}{E_c\ b\ (d - z)^2}\left[\frac{2}{9\ z} + \frac{1}{2\ pn\ d}\right].$$

Place the first derivative of the variables with respect to z equal to zero :

$$\frac{d}{dz}\left\{(d - z)^{-2}\left[\frac{2\ z^{-1}}{9} + \frac{1}{2\ pn\ d}\right]\right\} = 0.$$

Or, $-2\ (d - z)^{-3}\ (-1)\ \left[\dfrac{2\ z^{-1}}{9} + \dfrac{1}{2\ pn\ d}\right] + (d - z)^{-2}$

$$\left(-\frac{2\ z^{-2}}{9}\right) = 0.$$

Multiply through by $9\ z^2\ (d - z)^3\ \dfrac{pn}{d}$:

$$4\ z\ \frac{pn}{d} + \frac{9\ z^2}{d^2} - \frac{2\ pn}{d}\ (d - z) = 0.$$

Solve for $\dfrac{3\ z}{d} = k$, by collecting terms, and completing square :

$$\left(\frac{3\ z}{d}\right)^2 + 6\ pn\ \frac{z}{d} + (pn)^2 = 2\ pn + (pn)^2$$

$\dfrac{3\ z}{d} = k = \sqrt{2\ pn + (pn)^2} - pn$, as found by the author.

T-Beam, Neglecting Compression in Stem.

$$W_{dl} = \frac{1}{2}\ C \times \text{strain at } z \text{ from top} + \frac{1}{2}\ T\ \frac{f_s}{E_s}\ dl.$$

Strain at z from top $= \dfrac{f_c}{E_c}\ \dfrac{kd - z}{kd}\ dl.$

Also, substitute $f_s = \dfrac{T}{A}$; $E_s = n\ E_c$; $f_c = \dfrac{2\ kd\ C}{b\ t\ (2\ kd - t)}$.

$$W_{dl} = \frac{dl}{E_c}\left[\frac{C^2\ (kd - z)}{bt\ (2\ kd - t)} + \frac{T^2}{2\ n\ A}\right].$$

Mr. Williams.

Substitute $C = T = \frac{M}{d - z}$.

$$W_{dl} = \frac{M^2\, dl}{E_c\,(d - z)^2}\left[\frac{kd - z}{bt\,(2\,kd - t)} + \frac{1}{2\,n\,A}\right].$$

Now, $$z = \frac{3\,kd - 2\,t}{2\,kd - t}\cdot\frac{t}{3}, \text{ or, } kd = \frac{3\,t\,z - 2\,t^2}{6\,z - 3\,t},$$

$$\text{and } kd - z = \frac{6\,t\,z - 2\,t^2 - 6\,z^2}{6\,z - 3\,t}$$

$$2\,kd - t = \frac{-\,t^2}{6\,z - 3\,t}.$$

Substitute, and place first derivative with respect to z equal to zero:

$$\frac{d}{dz}\left[(d - z)^{-2}\left(\frac{-\,6\,t\,z + 2\,t^2 + 6\,z^2}{b\,t^3} + \frac{1}{2\,n\,A}\right)\right] = 0.$$

$$-\,2\,(d - z)^{-3}\,(-\,1)\left(\frac{6\,z^2 - 6\,t\,z + 2\,t^2}{bt^3} + \frac{1}{2\,n\,A}\right)$$

$$+\,\frac{(d - z)^{-2}}{b\,t^3}\,(12\,z - 6\,t) = 0.$$

Multiply through by $(d - z)^3\,b\,t^3$:

$$2\,(6\,z^2 - 6\,t\,z + 2\,t^2) + \frac{b\,t^3}{n\,A} + (d - z)\,(12\,z - 6\,t) = 0.$$

$$12\,z^2 - 12\,tz + 4\,t^2 + \frac{b\,t^3}{n\,A} + 12\,dz - 6\,d\,t - 12\,z^2 + 6\,t\,z = 0.$$

$$6\,z\,(2\,d - t) = 6\,d\,t - 4\,t^2 - \frac{b\,t^3}{n\,A}.$$

$$z = \frac{2\,n\,A\,(3\,d - 2\,t) - b\,t^2}{2\,n\,A\,(2\,d - t)}\cdot\frac{t}{3}.$$

This is exactly the same as the value obtained by the usual assumption, as can be shown by substituting the value of $kd = \frac{2\,n\,d\,A + b\,t^2}{2\,n\,A + 2\,b\,t}$ in the expression for z:

$$z = \frac{3\,k\,d - 2\,t}{2\,k\,d - t}\cdot\frac{t}{3} = \frac{6\,n\,d\,A + 3\,b\,t^2 - 4\,n\,t\,A - 4\,b\,t^2}{4\,n\,d\,A + 2\,b\,t^2 - 2\,n\,t\,A - 2\,b\,t^2}\cdot\frac{t}{3}.$$

Or, $$z = \frac{2\,n\,A\,(3\,d - 2\,t) - b\,t^2}{2\,n\,A\,(2\,d - t)}\cdot\frac{t}{3}, \text{ as before.}$$

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT

Discussion.*

By O. W. Peterson, Assoc. M. Am. Soc. C. E.

Mr. Peterson.

O. W. Peterson,† Assoc. M. Am. Soc. C. E. (by letter).—The irrigable area of the Sacramento Valley contains from 2 000 000 to 2 500 000 acres, and that of the San Joaquin Valley 4 800 000 acres.‡ The aggregate mean annual stream flow into the Sacramento Basin is, in round numbers, about 30 000 000 acre-ft., and the inflow into the San Joaquin is about 10 000 000 acre-ft. The northern basin has a run-off equivalent to a depth of 12 ft. over its irrigable area, that of the southern basin being equal to a depth of about 2.1 ft.

It is impossible, however, to conserve the entire run-off. The total capacity of available reservoir sites in the Sacramento Basin is sufficient to furnish a net annual yield of more than 3 000 000 acre-ft. This supply, in addition to the summer stream flow, will irrigate about 1 500 000 acres. The remaining lands can readily obtain a supply from return waters and from wells.

No definite information is available regarding the net yield to be obtained by practicable storage in the San Joaquin Basin. It would be most unusual if a net annual yield of more than one-half of the entire run-off could be conserved in reservoirs and underground storage combined. This would be nearly five times as great a ratio of storage capacity to run-off as that given for the Sacramento Basin. This 5 000 000 acre-ft., in addition to the summer stream flow, would supply

* Discussion of the paper by H. T. Cory, M. Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† La Grange, Cal.

‡ *Bulletin No. 239*, Office of Experiment Stations, U. S. Dept. of Agriculture.

Mr. Peterson.

about 1 800 000 acres, or somewhat more than one-third of the entire irrigable area in the basin.

The general conditions of water supply in the two valleys are fairly well known. At the recent Inland Waterways Convention, held in San Francisco, Senator James D. Phelan, California's Junior Senator, is reported to have stated the situation most aptly:

"We have before us the great Sacramento Valley—too much water, and the great San Joaquin Valley—too little water. This problem ought to be solved by science."

Now, regardless of this condition of affairs in the two valleys and of its realization locally, San Francisco is proceeding to develop for use outside the basin of the San Joaquin what is probably the best reservoir site in such basin, and to withdraw thereby a supply of water sufficient for the irrigation of nearly 200 000 acres of land. This is more than 10% of the entire area in the San Joaquin Valley which can be fully irrigated. Moreover, a very large proportion of such area is the higher-lying, rolling, foot-hill land adapted to citrus fruits, so that the present value of such a tract of 200 000 acres, with full but undeveloped water supply, is really not less than $30 000 000. The author's figure, $18 924 000, is entirely too low. In time, of course, the value would be much greater.

With this (at least) $30 000 000 added to the estimated cost to San Francisco of the proposed Tuolumne River development, it appears that a fairer basis of comparison with the sources in the Sacramento River Basin would be obtained.

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DEPRECIATION AS AN ELEMENT FOR CONSIDERATION IN THE APPRAISAL OF PUBLIC SERVICE PROPERTIES

Discussion.*

By J. N. Dodd, Esq.

J. N. Dodd,† Esq. (by letter).—The importance of this subject is well recognized, especially at the present time. Depreciation of properties, it is true, has always existed. Practically all properties become less valuable with age. Appraisals have been made since commerce began. Depreciation has always been considered as an element in such appraisals, in fixing the price which a purchaser should pay. However, not until lately have appraisals become of importance to the economic life of the community. As long as a business was subject to unlimited competition and the purchaser of a commodity was not compelled to buy at one place or from one firm, the amount at which the owner valued his business, or indeed for which he was able to sell it, was without influence on prices, and had very little effect on the community. Exchange of the property at too high or too low a figure was in no way a destruction of wealth. An attempt to capitalize the property at too high a figure and to compel the community to pay prices which would permit the prevailing rate of return on too high a valuation was fatal, as competitors were always ready to undersell. Competition, therefore, provided ample safeguards for the public. Mr. Dodd.

With the growth of modern public utility corporations, however, such safeguards no longer exist. These corporations deal with necessities, so that their products are those which the public must buy. Such corporations are virtually, if not actually, monopolies, and therefore the public must buy from them.

* Discussion of the paper by C. E. Grunsky, M. Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† New York City.

Mr. Dodd.

Moreover, it is provided by law that the corporation is permitted to earn a fair return on a fair valuation of the property. Therefore that valuation has an immediate and vital influence on the community, as it determines the amount which the community must pay for the commodity. For this reason, depreciation, and the extent to which it affects valuation, has become an important question during the last few years.

Definition of Depreciation.—As generally understood, depreciation may be defined as the loss of value of an article or property through age or through use. It includes, therefore, loss of value: through failure to give the original service; through increased cost of maintenance; on account of change in the demand; on account of the development of new and more economical machinery; and through the approaching date when the article or property must be replaced. All these causes affect vitally the value of the property.

The foregoing definition does not agree in all respects with the one given by Mr. Grunsky, who defines depreciation as:

"the lessening in worth of any perishable article or property. It is not, therefore, to be measured by inherent deterioration, due to wear and tear, but solely by a comparison between the probable life, the expectancy [the remaining life], and the cost of replacement * * *."

With the first part of the author's definition it is impossible to take exception. It is precisely the one always assumed in the appraisal of any property, whether owned privately or publicly, and whether the appraisal is for the purpose of purchase or of rate-making.

The second part of the definition, however, is open to criticism. It is difficult to understand how, with the first sentence as a premise, one can obtain the conclusion in the second sentence, as the author does.

Depreciation is a lessening in worth of a property, and it is due to several causes, not to one. An article may at any time show very little wear and accordingly have a very long remaining life. Yet, on account of changes in the methods of production, or in the demand for the product, the article is useless. The value of such an article, therefore, is lessened and the depreciation is high, but this lessened value is not on account of its short remaining life. The development of electricity for traction practically annihilated the value of the cable plants in New York City for the operation of street railroads, in spite of the fact that those plants had barely been installed, and, with ordinary usage, had many years of life ahead of them.

Necessity of Allowing for Depreciation.—If capital is to be invested in any business, two things must be assured, namely, security of the investment and a fair return on the capital.

Depreciation means a disappearance of the capital invested. Therefore, before capital can be secured for a project, there must be assurance that the probable income shall be such as to permit not

Mr. Dodd.

only a fair return on the investment but also full depreciation allowance.

Methods of Recovering Depreciation from Earnings.—Depreciation may be recovered from earnings in two ways: An amount sufficient to cover the depreciation loss may be returned from earnings to the investor, thus reducing his investment by an equal amount; or this amount may be reinvested in the business, thus keeping the capital up to its original amount.

In case of a business owned privately, it matters little which method is used. If the business is one which depends on a whim of fashion, it is well for the owner to try to reduce his investment at the earliest possible date by taking from the business the depreciation earned. If he wishes to remain in the business or to leave it on a firm financial basis, he will keep up the capital by returning to the business an amount equal to the loss by depreciation.

In case of a private business, it matters little to the community which method is used in recovering depreciation from earnings; whether the capital be lessened or whether it be retained at its full value.

In case of a business publicly owned, such as a stock corporation, and especially in case of a public utility company, it is usually imperative that the capital be maintained at its full value. Practically, the only exception to this is the case of a mine or like property. The value of a mine is decreased by the ore taken from it, and eventually the mine is valueless and the invested capital disappears.

In general, however, the possible investor must be fully protected against investing in a company which, under the guise of paying large dividends, has actually been repaying to the owners amounts which should have been credited to depreciation. Many of the scandals in the financial world are due directly to such practices. If the business concerned is that of a public utility company, the necessity of holding the investment intact is even more essential. The community has given the company a monopoly of the right to supply a certain commodity. In return for this grant, one of the rights of the community is continuity of service. The author admits (page 2809*) that "the 'obligation to replace worn-out parts' goes with every public service property." However, the only way to guarantee that the owner will replace such parts is to forbid him to withdraw from the property, even temporarily, the depreciation allowance earned.

In this connection, there are two statements of the author with which it is impossible to agree. On page 2806* is found the following:

"The amounts thus received [by the owner to cover depreciation] are usually held to be repayment of invested capital. This is a cor-

* *Proceedings*, Am. Soc. C. E., for November, 1914.

Mr. Dodd.

rect view when their aggregate amount without interest will in the life of an article amount to the cost of that article."

Apparently, the conclusion from this statement is that, if the owner receives in this manner 100% of the value of the article, the full value must be written off the capital; but if he receives only 99% of the value of the article, the capital remains at its full original amount.

The author defines "depreciation" as a "lessening in the value". A corresponding amount, therefore, should be written off the capital simultaneously with the depreciation. It is impossible to reach any other conclusion than that the amounts received by the owner to cover depreciation are repayments of invested capital, whether he receives the entire amount or only a part.

Again, on page 2819,* he says that depreciation or amortization repaid to owners can be written off the invested capital only when one of three methods of calculating depreciation has been followed, and that the depreciation thus calculated has been earned. It follows from this that, if any owner should sell part of his plant by regular or irregular installments, or if for any other reason the value of the plant be decreased, the amount of such decrease cannot be written off the capital unless he also receives regularly in a given manner corresponding sums from earnings.

Manifestly, it is impossible to agree with this conclusion. Depreciation is real. Unless the loss is restored, it must be written off capital at once. The sums received by the owner to cover depreciation are repayments of capital, regardless of the law followed, and indeed regardless of whether any law is followed.

Various Methods of Computing Depreciation Allowance.—The author has described a number of ways of computing the correct depreciation allowance. Some of these, as defined by him, consist of a repayment from earnings of the investment, or as he terms it, "of an amortization of capital". Others are 100% methods in which there is no amortization of capital, but the investment is kept at its original value by the reinvestment from earnings of an amount computed to be equal to the depreciation loss.

As pointed out previously, there should be no repayment of capital in case of a public utility company. Accordingly, no method involving such repayment need be considered in connection with appraisal for rate-making. The author gives two methods of calculating the depreciation on a property and the amount of earnings that must be set aside each year for replacement in such a way as to keep the capital intact. One he calls the "Sinking-Fund Method". According to this, the depreciation allowance is the amount which must be set aside annually in a sinking fund at compound interest so that the fund

* *Proceedings*, Am. Soc. C. E., for November, 1914.

at the end of the life of the property will be equal to the cost of replacing the property. At first sight, this appears to be a very logical, if not the most logical, method. Mr. Dodd.

The second "100% method" suggested is what the author calls the "Unlimited-Life Method". According to this, the owner receives from earnings the annual depreciation allowance, but morally binds himself to replace the article at the expiration of its life. The amounts thus received by him must be set aside in a fund which must be used, principal and interest, for no other purpose than the replacement of the property at the end of its life. As the owner is not entitled to use this fund for any purpose, he is entitled to receive, from the business, interest on the original value of the investment.

Unfortunately, a better law is needed in business than the moral law. An absolute essential is that the sums thus set aside shall be under the control, not of the individual owner, but of the company itself. As pointed out before, no method should be permitted which allows, even temporarily, the withdrawal from the business of the depreciation allowance.

What is called the "Straight-Line Method" consists in setting aside from earnings an amount each year equal to the replacement cost divided by the probable life in years. According to the author, these sums are repaid to the owner, and this, therefore, is not a "100% method". As a matter of fact, however, it may be used as a "100% method", if these sums be retained in the company and re-invested either in the business or otherwise.

A criticism which has been made against the "Straight-Line Method" is that it requires the earnings to be as heavy during the early years as during the later ones, when the business is more fully developed and the payments can be more easily met. This may be a valid criticism. However, the depreciation is computed, not on the plant as a whole, but on the detailed parts, and usually, in case of an appraisal, most of these detailed parts actually included in the appraisal are provided after the plant is developed, and certainly the first years of the life of a machine are generally the most efficient and can best afford the heaviest depreciation charges.

In the Report of the Special Committee on Valuation, entitled "Valuation for the Purpose of Rate-Making", a method of computing depreciation was suggested which was designed with a special view to the criticism of the "Straight-Line Method" just quoted, and provided for increasing payments with age. Such a method is open to three criticisms: It provides for the heaviest payments during the latest and most inefficient years of the life of the property. It adds tremendously to the accounting difficulties. Imagine, for example, the labor of keeping the depreciation accounts on, say, 100 000 telephone poles divided by age into groups, no two of which are numerically

Mr. Dodd.

equal, the original cost of the poles varying widely and the depreciation rate on each group differing from the rate on any other group. In this respect, either the "Straight-Line Method" or the "Sinking-Fund Method" has the advantage of great simplicity, as each provides that the annual depreciation allowance is merely a certain percentage of the sum of the cost of the replacement of all the poles. The method proposed is practically impossible of application except for a few units. Such units would naturally be large ones.

The third objection to this method is that it is not one of approximating the actual depreciation at any time, but only a formula for building up a fund which will probably equal the depreciation at the end of a certain period.

It is true that this last criticism applies also with equal force to the preceding methods of calculating depreciation. However, it serves as a strong argument against a method which is much more difficult than any other and, at the same time, is no more accurate.

Indefiniteness of the Problem of Determining Depreciation.—Depreciation of a property is real. Almost every item in the inventory is subject to it on account of one or more of the causes given in the definition, and the total depreciation may, and often does, attain a very high proportion of the original value of the property.

Nevertheless, methods of calculating depreciation are essentially indefinite, and must be suited to the conditions of the case. The life of an article varies, of course, with the usage it receives, with the demands on it, and with the character of the maintenance; but, even under the same care and the same conditions of service, the life of two pieces of apparatus apparently identical in construction and material will differ greatly from each other. In other words, actual depreciation cannot be predicted beforehand.

More than this, it is impossible to determine the actual depreciation which has occurred. It is seldom that one can find two people to agree on the value of an article: as measured by its remaining life; as measured by the service it can render; as measured by its increased maintenance charge; or, in fact, as measured by any of the elements that affect the depreciation. This being so, it follows that no hard-and-fast rule can be observed in calculating depreciation. When accounts are begun, a method should be adopted which one has reason to believe will be approximately correct. The method, however, and the amounts periodically set aside must be changed from time to time as the condition of the property dictates. It is impossible, therefore, to agree with the author when, on page 2820;* he insists that once a method has been adopted, it must never be changed.

Typical Method of Allowing for Depreciation.—In order to get a clear idea of the principles underlying valuations, it is advisable to

* *Proceedings*, Am. Soc. C. E., for November, 1914.

consider briefly the accounting method followed by a typical firm, conservatively managed, which tries to keep its capital up to the original value. A depreciation fund is built up by crediting to it from earnings amounts equal to the estimated losses in value of the plant during the year. This loss in value is the loss as considered from all points of view, not merely the loss as measured by its life, or service value, or any other of the single elements going to make up its value. Mr. Dodd.

If untouched, this depreciation fund plus the remaining value of the plant should be equal to the original investment. However, the fund is not usually untouched. It matters not to the investment whether the fund is allowed to accumulate, whether it is invested, or whether it is spent for betterments or for extensions. In any case, the value of the plant is: The depreciated value of the original plant, plus the amount in the depreciation fund, plus the depreciated value of the extensions and betterments. If the methods of estimating are correct, the total value at any time is not less than the original investment.

Most companies are constantly extending their property, and the amount in the depreciation fund is usually spent for extensions. When the items must be replaced at the end of their life and there is not sufficient in the depreciation fund, the amount may be obtained by a bond issue or increased stock based on the additions to plant.

When accounts are kept in such a manner, it becomes difficult to credit any interest to the depreciation fund. The fund being reinvested in the plant, the interest is treated as earnings. The amount credited to the depreciation fund varies from year to year as the condition of the plant dictates. From this may be seen the value of the "Straight-Line Method". It is simple. It is at least as accurate as any other, and adjustments can easily be made from time to time as conditions dictate.

Theories of Valuation.—Many of the author's statements, to which exception has been taken, are, perhaps, based on a certain theory of valuation which he gives. On page 2818,* the statement is made:

"There can be no fairer nor better way of determining the earnings of a public service property than that under which the owner is regarded as the agent of the public".

On page 2817,* are the words:

"When an appraisal is to be made for rate-fixing purposes, of a plant which has continuously been operated with inadequate earnings, there is no escape from the conclusion that the investment has remained undiminished".

Difficult as this latter statement is to accept, the truth of it must be admitted if the first statement is granted. If the owner is an agent

* *Proceedings,* Am. Soc. C. E., for November, 1914.

Mr. Dodd.

and erects the plant purely for the service of the community and as the agent of the community, clearly he is at all times entitled to a return on the original investment, whatever may be the condition of the plant. Such a theory has been strongly presented by others. However, the fact remains that the community does not engage the owner to erect and operate the public utility plant. Such a theory brings the certain conclusion that if an owner makes a mistake and erects a plant not suited to the needs of the community, the community is nevertheless compelled to pay him full interest on his investment. Desirable as such conditions might be to the investor, it breaks every economic law; it nullifies the exercise of sound business judgment; it discourages the ingenuity of manufacturers in developing better types of machinery, and penalizes the community.

The rules governing the installation and operation of a public utility company should differ very little from those governing any other company. The community gives the company a monopoly in a certain trade. This should produce a certain income. The owner of the property has the opportunity of judging what this income should be and of erecting a plant accordingly. He has no right to expect the city to pay for his mistakes.

On the other hand, the city has the right to good service, to all the improvements in the art, and at prices no higher than if it is protected by competition. In other words, depreciation due to any and every cause should be taken account of in the valuation just as mercilessly as if competition were in full force. No other plan is economically correct.

Conclusions.—Depreciation is the lessening of the value of a property for any cause, whether due to decreased efficiency of the plant, to development of improved machinery, to change in demand, or to the approaching date of retirement.

This loss is real, and, unless replaced, must be written off the capitalization.

The amount of such loss is not susceptible of exact statement or calculation. It cannot be foretold with correctness, and, when it has occurred, its amount at best can be only approximated.

The owner of a property has a right to expect earnings sufficient to cover the loss of capital due to depreciation, in addition to a fair interest on the investment.

In a public utility company, the amount so earned to cover depreciation loss must be reinvested in the business.

For all practical purposes, the "Straight-Line Method" is perhaps the best for calculating depreciation. However, it must be used with the understanding that it is only an approximation, and the total amount must be adjusted from time to time as the condition of the plant demands.

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THE WATER-PROOFING OF SOLID STEEL-FLOOR RAILROAD BRIDGES

Discussion.*

By Messrs. A. H. Rhett and Philip B. Walker.

Mr. Rhett.

A. H. Rhett,† Assoc. M. Am. Soc. C. E. (by letter).—In any discussion of the subject of water-proofing, it seems inevitable that, sooner or later, three points of controversy will present themselves, namely, felts *vs.* fabrics; non-adhesion *vs.* adhesion; and thickness *vs.* thinness.

Felts vs. Fabrics.—It is unfortunately true that where a bridge structure is submitted to a variation of temperature of from, say, zero to 120° Fahr., its water-proofing membrane is subjected to the same disrupting strains at the low as at the high temperature. It is generally conceded to be true also, that for a membrane to perform its function successfully, it must not only be water-proof, but elastic, and as it is to be subjected to the same strains at low and high temperatures, it must, if elasticity is an essential requisite, be as elastic at the low temperature as at the high one.

Now, in either the felt or fabric membranes, as commonly used, the plying medium is a compound, and although the felt or fabric, as the case may be, is more or less elastic at all temperatures, it is the writer's experience, that the plying compounds are not only very inelastic at temperatures below freezing, but very brittle and friable.

It would seem inevitable, therefore, that at such temperatures the compound will be fractured by the deflection of the structure and impact of the load, and when thus once broken, the disrupted segments can certainly not be re-united until the structure is once more subjected to a temperature equal to that of the melting point of the compound,

* Discussion of the paper by Samuel Tobias Wagner, M. Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† Brooklyn, N. Y. Previous discussion of this paper by Mr. Rhett was printed in February, 1915, *Proceedings*.

Mr. Rhett.

which temperature is usually specified as much higher than any to which the structure is ever likely to be subjected.

The membrane, then, after having once been subjected to the low temperature will consist thereafter, presumably, of a number of sections of inelastic compound held together by the felt or fabric. As the saturating element of the felt is kindred in nature to the plying compound, the felt will certainly lose elasticity at the lower temperatures, but if its coherent quality is sufficient to withstand, without impairment of its water-proofing integrity, the strain that disrupts the compound, then, of course, it will bridge the break in the compound and bar the passage of water on a return to the higher temperatures. With the fabric membrane, the case would seem much worse, and it is very difficult to comprehend, how, after the compound is once broken, a few strands of cotton or burlap can prevent the passage of water through the membrane.

Non-adhesion vs. Adhesion.—When a membrane is cemented to the surface to be water-proofed by a compound that is rigid and inelastic at low temperatures, it becomes evident that the elasticity of the reinforcement cannot come into play at those temperatures until both that surface and the compound have broken, and that then the effective elasticity is localized to the line of rupture. As far as the integrity of the membrane is concerned, then, it would seem that the membrane should not be cemented to the water-proofed surface, though the possibility of the water getting under it, in case of puncture, when not so cemented is certainly a strong argument in favor of doing so.

Thickness vs. Thinness.—If a stratum of material is truly and inherently water-proof, how can it be made more so by multiplying it? If each stratum is defective, then, on the theory that the defects in successive strata may be staggered by the multiplication of the strata, there may be some basis for argument. The writer believes that where a material is truly water-proof, any increase in thickness beyond that necessary to cover effectively the surface to be water-proofed, brings with it a very injurious effect in the proportionate decrease in flexibility, and certainly a very undesirable one in the proportionate increase in cost.

Mr. Walker.

PHILIP B. WALKER,* ESQ. (by letter).—The writer has read this paper with much interest. The question of water-proofing is of vital importance in modern engineering structures and worthy of careful study and investigation. The writing of satisfactory specifications which will cover this branch of construction will be heartily welcomed by the members of the Profession.

The writer wishes to commend the author for the stand he has taken in not specifying coal-tar or any of its products for use in

* Asst. Engr., Boston Transit Comm., Boston, Mass.

water-proofing operations, as he is firmly convinced that this is not a proper material to be used in the construction of an elastic water-proof membrane. Its brittleness at temperatures less than freezing and its fluidity at summer temperatures, as well as its flow under pressure, make it a material on which no reliability can be placed for water-proofing purposes. The writer has not yet found, by experiment or in practice, that exposure to water affected asphalt any more than it did coal-tar. The difference in price between these products is more than made up by the smaller quantity of asphalt used in covering a given surface in order to get the same water-proofing effect. Mr. Walker.

As a reinforcing medium for the membrane, the writer would recommend only a suitably treated fabric. He has found by experience that felts of any type, or burlaps, are wholly unfit to be used in this connection. Less than a month ago he had occasion to examine a piece of water-proofing which had been in place about 9 months. A burlap which, after experiment, had been chosen as the best of several makes was used. The membrane consisted of two plies of this burlap with three coatings of oil asphalt. Poor workmanship in the formation of a lap had resulted in admitting moisture to the burlap, which had absorbed this moisture by capillary action and, as a result, a large section of this material had rotted. Felts have been known to do exactly the same thing. In fact, any material which is simply coated and not saturated with a water repellant is unsuitable for use in a water-proofing membrane.

The base of the fabric used should be a closely woven drill weighing about 12 oz. per sq. yd. This should be completely saturated with an asphalt similar to that used in forming the finished membrane. Not every asphalt can be used with a given fabric, so experiment should determine the proper one. Although each individual thread forming the fabric must be completely saturated, the space between the threads must be open to allow the successive moppings of asphalt to unite. The drill used should be woven so as to have equal tensile strength lengthwise and crosswise.

A membrane made up of such a fabric will have considerable stretch and will not crack or tear even under severe temperature changes. The use of such a fabric permits of tight joints around gussets, stiffeners, etc.

In regard to the asphalt the writer would consider that a smaller range in the melting-point might be specified. A difference of from 20 to 25° Fahr. between the maximum and minimum melting-points ought to cover the various conditions as found on the work and yet give an asphalt less variable in those qualities which the melting-point controls.

In specifying the melting-point, he would suggest that the author give in detail the method of determining it, until a standard method

Mr. Walker. is accepted. When stating the method of application and the quantity of asphalt to be used, it would be more complete if the following were included: "Asphalt shall not be heated at any time to a temperature higher than 450° Fahr. The temperature of the asphalt in the pail just before being mopped into the fabric shall not be below 380° Fahr."

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RECONSTRUCTION OF THE NORFOLK AND WESTERN RAILWAY COMPANY'S BRIDGE OVER THE OHIO RIVER AT KENOVA, WEST VIRGINIA

Discussion.*

By L. L. Jewel, M. Am. Soc. C. E.

Mr. Jewel.

L. L. Jewel,† M. Am. Soc. C. E. (by letter).—There is perhaps no field of engineering endeavor in which boldness, tempered with sound judgment, reaps such rich rewards as in the erection of structural steel. This quality of intelligent daring is particularly demanded in the erection of heavy modern bridges, the size and weight of their members alone making their transportation and assembly a difficult problem. When to this is added the uncertain action of a treacherous stream, and the necessity of maintaining clear paths for traffic both above and below, conditions are created which demand a very thorough knowledge of engineering principles and a high order of executive ability; and these, perhaps, are not always fully appreciated by the purely designing engineer. The authors have given a very clear and concise description of such an undertaking, and of the methods by which it was brought to a successful completion.

The study of such a paper or problem will be along four lines of primary importance, namely, risk, cost, time, and interference with other operations. As all these considerations are largely interdependent and matters of judgment rather than fact, there is generally ample room for wide difference of opinion. The case described by the authors is not an exception to this general rule in regard to certain parts of the work.

* Discussion of the paper by William G. Grove, Esq., and Henry Taylor, Assoc. M. Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† Wytheville, Va.

Mr. Jewel.

It is unlikely that any one familiar with erection costs will disagree with the authors' statement that the two shore spans, Nos. 1 and 5, could have been erected at less cost by the method of straight falsework and a gantry traveler; nor does it seem open to question that this cheaper method would have also taken less time, and, to that extent at least, would have lessened interference with the railroad traffic. There is, however, some question in the writer's mind whether this cheaper method would not have really involved less risk—the most important consideration of all—than the method used.

The deflection of these two spans under their regular traffic load is given in Fig. 28, as $\frac{7}{8}$ in. and $1\frac{1}{2}$ in. below the dead-load camber for Spans 1 and 5, respectively, the great difference being due no doubt to excessive wear in the pin-holes of Span 5. The maximum deflection of Span 1 under its erection load is given by the authors as "about $2\frac{1}{2}$ in. below the dead-load camber, or $\frac{1}{2}$ in. below a horizontal line drawn from center to center of the end pins", which is a little more than is shown in the theoretical diagram, Fig. 22. A comparison of the erection-load and the live-load deflections for Span 1 is $2\frac{1}{2}$ in. to $\frac{7}{8}$ in., or nearly 3 to 1. At this point in the process of loading the old span with the new one, no top chords or end posts had been placed, and, moreover, the old span had been relieved of the weight of its own track and track stringers.

It would seem, therefore, that with the falsework carrying the old track stringers and track, and also a more or less indeterminate part of the weight of the new work, the old span was loaded very nearly to the line dividing safety from risk. It was carrying a uniform load of about 7 000 lb. per lin. ft. of new steel, the weight of two material tracks, the brackets supporting them, the erection blocking, hangers, etc., and the traveler track and its falsework—a uniform load approximating 8 500 lb. per lin. ft. In addition to this uniform "dead" load, it carried the erection traveler and the weight of the heavy chords, while being placed, concentrated at two panel points. Such a traveler hoisting, swinging, lowering, and suddenly stopping heavy loads is certainly the liveliest kind of a "live" load, and, on occasion, its total weight may be carried largely by only one truss.

Taken altogether, this was certainly a very severe load for a span designed for Cooper's E 40, 25 years ago, and in hard service for 21 years. If, during erection, a rise in the river should have destroyed the falsework—which was merely an auxiliary to erection—an addition of about 2 000 lb. per lin. ft. would have been made to this already heavy load. However, the greater part of this addition, that is, the weight of the top chord, would have extended only from one end up to, and including, the position of the traveler.

Additional value would have been given to an already valuable and purely erection paper, if the authors had stated the assumed load-

ing, including traveler concentrations, for these two old spans, and the unit stresses involved, instead of, or in addition to, the stress sheets for the new spans. It is, of course, very desirable to be far more liberal as to unit stresses for a temporary purpose, such as the comparatively short period of erection, than for a permanent structure of the same kind, but there is always the question as to just how far this liberality should go. The writer has allowed a stress practically equal to the elastic limit of the material in certain heavy erection equipment, but only when such stress could occur a very limited number of times, at long intervals, and for only a few minutes at a time. Moreover, the material was new and subjected to the same rigid inspection and tests as the material for the permanent structure. Judging merely from a comparison of the deflections given, a reasonable assumption of the traveler's weight, and the loading for which these old spans were designed, and without making any calculations, it would appear that such a stress was exceeded perhaps in the present case. Mr. Jewel.

As to the risk involved in depending on falsework for the entire erection load as well as for traffic, some doubt may be expressed whether there has ever been a flood in the Ohio River, barring ice, which would have destroyed well-constructed falsework, of the type necessary for this erection, under the two shore spans. At any ordinary stage of water, all of Span 1 is practically over dry land, and at no stage is there a current under this span which could be considered very dangerous, either from its direct force or scour—a condition due partly to the wide expanse of low flat land on the eastern, or rather southern, side which allows extensive spreading and also, of course, to the distance of Span 1 from the line of principal flow. Span 5 does not have so much advantage of location, but it is also some distance from the center of the channel and well out of the maximum current at any stage of the water. Ice can hardly be the serious consideration here that it is farther north (in the Pittsburgh District), and, in any event, it goes out in the spring for all the rest of the year.

In expressing a preference for the straight falsework method over the one used for the two shore spans, the writer does not forget that "nothing succeeds like success" and that the method used has been proved out. At the same time, he believes that many of our larger rivers bear a blanket reputation for deviltry without any consideration of the fact that every stream's character changes very materially with the topography of its valley. Many of our "bad" rivers are also charged with numerous wrecks due to absolutely avoidable conditions or causes. Every one interested in erection work has observed the use of falsework, for example, which could be more accurately classified as a gambling device with the chances against the contractor, than as an engineering structure. Without reference to the case under

Mr. Jewel.

discussion, it may be said that every erection enterprise deserves individual study and decision, unprejudiced by records of disaster at other locations.

Another point which seems worth mentioning for comparison is an alternate method for delivering material for the three spans erected by the cantilever method. Apparently, there was sufficient depth of water for a barge, because all floor-beams were lowered from the bridge to a barge and turned so as to be hoisted by lines on each side of the bridge. The question naturally arises, why was not all material delivered directly from yard to traveler by barge and tugboat? A considerable percentage of the 1 300 tons of special erection material would have been saved by omitting the material track brackets, as well as considerable labor in building and dismantling the tracks. By the barge method, all new truss material would have had one less, and all new floor material two less, handlings; and it would appear from the photographs that almost any stage of water which would hinder the operation of the tugboat would flood the material yard and so stop operations in any event. This alternate method is suggested only on the assumption that there could never be lack of water to float the barge under the traveler.

The cantilever portion of the work is very interesting. Although it shows nothing entirely new, it is the heaviest simple truss work erected by this method that has come under the writer's observation, and he would like to express appreciation of the great mass of details which required accurate working out in order that all clearances and the closure of the channel span could be so well provided for. The authors' statement that the 518-ft. trusses are the largest riveted trusses ever built, may be the case if over-all dimensions only are considered. Much heavier simple truss spans, which are riveted throughout, are the three channel spans of the Union Depot bridge and Terminal Company's bridge over the Missouri River, at Kansas City. One span, which carries a lifting deck to give clearance for navigation, is 425 ft. 6 in. from center to center of end bearings, and contains 4 823 tons of steel, including machinery. The two fixed spans are 423 ft. 0 in. from center to center of end bearings. Each contains 4 008 tons of steel, about $9\frac{1}{2}$ tons per lin. ft. They are double-deck spans carrying a double-track railroad below, and street-car lines, carriageways, and sidewalks above. The bridge was designed by Waddell and Harrington, Consulting Engineers, and was fabricated and erected by the McClintic Marshall Construction Company, Paul L. Wolfel, M. Am. Soc. C. E., Chief Engineer. The writer was, at that time, Manager of Erection for the contractors, and was responsible for the erection plans and methods. The field work was completed under the management of E. A. Gibbs, Assoc. M. Am. Soc. C. E., and W. R. Hughes, Jr., Assoc. M. Am. Soc. C. E., was Resident Engineer.

Mr. Jewel.

The erection presented no unusual features except the great loads involved, several members in each span weighing considerably more than 100 tons, and the fact that a wedge device operated by a screw was used under each panel point in place of the usual crude wooden wedges. In this manner, it was not difficult to assemble the bottom chords in a level line, and thus all the chord splices, which had been assembled previously and drilled, were made practically perfect. After they were riveted, the wedges were lowered the proper distance at each panel point, working from the center to each end, and the massive chords were quickly, easily, and accurately brought to the position of no stress. These same wedge devices were used by the writer to lower the Panama lock gates over their pintles.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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SUGGESTED CHANGES AND EXTENSION OF THE UNITED STATES WEATHER BUREAU SERVICE IN CALIFORNIA

REPORT OF A COMMITTEE OF THE SOUTHERN CALIFORNIA ASSOCIATION OF MEMBERS.

Discussion.*

BY N. C. GROVER, M. AM. SOC. C. E.

Mr. Grover.

N. C. GROVER,† M. AM. SOC. C. E. (by letter).—The writer is in hearty accord with the suggestions made in this report for rendering the climatological work of the United States Weather Bureau of greater practical value to the communities and industries of California. The desirability of collecting systematic records of temperature, precipitation, accumulation of snow and its equivalent in water, has long been recognized by Weather Bureau officials, as well as by others who need records of that character. The difficulty, as the writer understands it, however, is in devising instruments that will record correctly the phenomena desired, without daily attention, for it is unusual to be able to obtain reliable observers at high elevations in the mountains. Without such resident observers, or instruments that can be relied on to record the data without attention for several weeks, the cost of collecting the records becomes practically prohibitive. There is no doubt, however, that the officials of the Weather Bureau will gradually develop methods by which satisfactory records may be collected at high altitudes.

* This discussion (of the report by George S. Binckley, M. Am. Soc. C. E., and Charles H. Lee, Assoc. M. Am. Soc. C. E., published in February, 1915, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Washington, D. C.

Mr. Grover.

The following statements (pages 250 and 251*) seem to be likely to be misinterpreted:

"In such regions the gathering of climatic data, as a basis for the study of stream flow and water supply, should be carried on in mountains within the productive portions of drainage areas.

* * * * * * * *

"The practical usefulness of climatic data is in forming a basis for the forecast of future stream flow variation. It is on the latter information that the magnitude or practicability of irrigation projects, hydro-electric power systems, municipal water developments, flood-protection works, or any other large engineering work, depending on water supply or stream flow, is based. Such data have also great value in the design of engineering structures for the control of water, such as dams, spillways, head-works, bridges, levees, etc."

Though the statements are not definite in this respect, the reader is left to infer that climatic data collected at properly located mountain stations will yield all the information as to water supply and its variations needed for the design of the important works mentioned. The dangers attending the use of estimates of water supply based on records of precipitation have been forcefully demonstrated by the many failures of hydraulic works of all classes. Such failures have been especially numerous in the irrigated areas of the West, and should serve as an object lesson not to be forgotten, but, unfortunately, many engineers appear never to have learned the lesson at all. Therefore, the inference that might be drawn from the report is particularly fallacious, in view of the reliance that many engineers, both in the East and the West, continue to place on estimates based on records of precipitation, even though it has been demonstrated again and again that such estimates are not only not dependable, but positively misleading.

* *Proceedings*, Am. Soc. C. E., for February, 1915.

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CINDER CONCRETE FLOOR CONSTRUCTION BETWEEN STEEL BEAMS

Discussion.*

By Messrs. Myron S. Falk, Guy B. Waite, Albert Oliver, Emile Reed Low, Arthur H. Diamant, Ira H. Woolson, J. R. Worcester, and Gardner S. Williams.

Myron S. Falk,† M. Am. Soc. C. E.—The authors deserve much credit for the very valuable experiments they have made on the physical properties of cinder concrete. They have again demonstrated how extremely variable those properties are, and, in spite of these facts, the experiments show how safe the use of this unreliable material has been in the City of New York, where the construction of such beams in the field is far from realizing the conditions obtainable in the laboratory. The speaker has seen cinder concrete laid in freezing weather, subjected to alternate thawing and freezing, and he has seen it laid in driving rain, continuous for two or three days after laying, and yet in none of these cases has he seen failure. Mr. Falk.

The speaker doubts whether reinforced cinder-concrete beams, as ordinarily used for building purposes in the City of New York, are subject to any rational theoretical design. That the basic ideas on this subject are indefinite may perhaps be best noted by the fact that, in this discussion, various speakers have used indiscriminately the words "arches", "slabs", and "beams". As a matter of fact, practice shows actual conditions to be more complicated than any one has yet suggested, as the exact status may be described as a combination of arch, beam, and slab supported at four edges; and, further, in actual field practice, the load is never placed directly on the beam itself, as shown in the tests described by the authors, but is distributed on the beam

* Discussion of the paper by Harold Perrine and George E. Strehan, Juniors, Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† New York City.

Mr. Falk.

(when wooden floors are used) by longitudinal sleepers covered with one or two thicknesses of flooring, or by a plain concrete slab, not reinforced, when cement-finished floors are used.

The speaker has no doubt that in many cases loads in buildings are carried to the steel frame without any stresses being developed in the cinder-concrete beams.

The Building Department of the City of New York acknowledges that, in certain cases of floor construction, arch action exists, and that tie-rods are required between the floor-beams to take care of the thrust. In other cases, it acknowledges that there is no arch action, or rather that the form of construction is safe without considering arch action, by permitting tie-rods to be omitted. In other words, cinder-concrete floor construction, as practiced in New York City for ordinary spans of from 5 to 8 ft., is not subject to rigorous design; this fact is emphasized when it is attempted to determine by theory the stress conditions in the steel reinforcement of test beams; as stated by the authors, impossible stresses of 300 000 lb. per sq. in. might be found to exist in the steel.

The speaker, in his practice, has had much difficulty in determining the reasons for the appearance of slight cracks in the ceilings of buildings containing floor construction designated by the authors as "flat-ceiling" or "bottom-flange construction". The usual attempt to explain these cracks is to say, offhand, that they are caused by expansion or contraction, or that the small quantity of concrete on the lower flange of the supporting steel beam causes the trouble. These explanations do not reach the seat of the trouble, and it would be advisable to have a thorough investigation to see whether the faultiness in this kind of construction cannot be eliminated.

Mr. Waite.

GUY B. WAITE,* M. AM. SOC. C. E.—Accurate data, based on tests of commercial cinder concrete, have been needed for many years, and this paper is undoubtedly one of the most important contributions to this field of engineering. The Engineering Profession should feel indebted to each and every one who has given time, talent, and money to produce these results.

The past and present manner of using cinder concrete in and about New York City, based on load tests, was good enough in the "old days", before any kind of reinforced concrete had been standardized. Since about 1900, reinforced stone concrete has been designed almost exclusively in accordance with the principles of the common theory of flexure.

Many of the larger cities have also put cinder concrete floor slabs, between steel beams, on the same basis for design as the stone concrete, using, however, lower working stresses for the former.

* New York City.

In their deductions, the authors recommend 30 as the ratio of the moduli of elasticity of cinder concrete and mild steel, and 300 lb. per sq. in. as the extreme fiber stress for the cinder concrete in the slab. Mr. Waite.

The authors, therefore, have substantially confirmed the unit stresses for cinder concrete which were adopted some years ago by Philadelphia, Baltimore, Boston, and Chicago, for the design of cinder concrete floor slabs.*

Although all cities have adopted definite unit stresses for the design of stone concrete, many, including New York City, do not recognize any unit stresses for cinder concrete.

The authors have pointed out the variableness of cinder concrete, which is very much greater than that of stone concrete. As cinder concrete is less reliable than stone concrete, it would seem that its design according to some definite standard would be more important than for stone concrete.

The approvals for the cinder concrete constructions now in use were based on tests of uniformly distributed loading over a sample slab. This sample was constructed as carefully as skill could make it; and the loading was placed so as to cause the least strain on the slab. Within the last few years, since Mr. Strehan has had charge of the testing for the City of New York, concentrated loading has been substituted for the former distributed loading, but, unfortunately, approvals from such loading gave so much lower results that those from the former tests are still the only ones in general use.

Before the change in the manner of testing, the system had developed into an art of competing to see who could go the farthest with the thinnest slabs, having the maximum spans, and with the greatest loads. So great became the proficiency that, toward the latter days, slabs with 3 in. effective depth and 8-ft. spans were approved for 250 lb. per sq. ft. The natural result of this form of testing was to surround cinder concrete construction with a kind of mystery, not existing in stone concrete of similar form.

The particular form of reinforcement used in the mysterious construction was supposed to possess the magic power necessary to overcome the action of gravity and make the principles of mechanics, which act on stone concrete, inoperative in the case of cinder concrete. For several years the commercial interests which have been benefited by this chaotic condition have resorted to all kinds of methods to have a new Building Code passed which would arbitrarily specify cinder concrete in such a manner that their particular kind of reinforcement would have preference over all others.

Generally, stone concrete slabs, under the same conditions as the cinder, are designed, and their constructions superintended, by

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1785.

Mr. Waite.

engineers; but in the case of cinder concrete used in accordance with the original test, nothing is required except a promise to construct in a manner similar to the tested construction. The result is that Mr. Tom, Mr. Dick, and Mr. Harry, who have no knowledge of engineering, are doing most of this kind of work.

The tested slabs for these approvals were made with restrained ends, but those in buildings may be almost anything.

The authors have chosen for their tests ideally restrained and ideally free ends, the two extreme kinds of slabs which are the antitheses of one another. There was only one slab tested, called "special", which was between these two extremes. Therefore, the present investigation supplies no data tending to throw new light on the conditions in buildings where ends are only partly restrained or only partly free. In the average building about 25% of the floor area consists of outside slabs, next to walls or openings in floors.

This series of tests contains data concerning slabs made continuous (similar to construction tested and approved by loading) by stretching wire mesh over the supports and putting it under tension by stapling it down to the wood forms; moreover, in these tests, the concrete is carried 1 in. above the tops of the steel beams, thus entirely encasing the reinforcing wire mesh.

This form of construction, however, is not what goes into the floors of buildings. In most floors, electric and other conduits and floor sleepers are run crosswise of the steel beams, resting directly on the top flanges. This is partly for the convenience of having substantial level surfaces on which to lay conduits, and partly because, if raised higher, head-room would be taken from the floor.

The actual process of laying the wire mesh reinforcement in buildings is to roll it across the entire floor, allowing it to spread loosely over the tops of the steel beams. The speaker has never seen a single case where the reinforcement was brought tight over the flanges of the steel beams by stapling it down to the wood forms. The reinforcement is further loosened by the tamping of concrete. Thus, it is left loose and exposed over the flanges, instead of being embedded in concrete, as in the tests. Therefore, in practice, the slabs are only restrained to the extent of the initial tension in the reinforcement.

If there is play in the reinforcement over the beams, the end restraint will take place only when the slab has failed sufficiently to bring the end reinforcement into tension.

Further, in the bottom slab construction, a part of the slab is entirely below the steel beams, and the reinforcement is more or less exposed. The wire mesh which straddles the steel beams is often trampled up against the web, to get it out of the way of the concrete. Figs. 45 and 46 show the top and the bottom types of construction as actually built in the floors of most buildings.

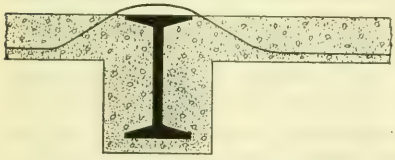
Mr. Waite.

Some years ago the speaker proposed to use a continuous reinforcement over the tops of steel beams, for similar building construction, but, after trying it on one job, he gave it up as impractical, on account of the objections referred to. Therefore, his opinion is that the restrained slabs tested by the authors do not represent the conditions in the average building.

The free-end slabs were tested by the authors with rollers at the points of support, a condition not found in actual floors. The data obtained are too severe in the case of the free-end slabs and not severe enough in the case of the so-called fixed-end slabs.

If the intention of the authors is to start with ideal conditions and later work toward practical constructions, this should be made clear, so as not to confuse the uses for the deduced formulas, which are not applicable to present building constructions.

The authors find that one empirical formula is applicable to the free-end slab and an entirely different one to the restrained slab. Also, they find that the restrained slab is capable of carrying two and one-



TOP SLAB,
CINDER CONCRETE BETWEEN
STEEL BEAMS

FIG. 45.

BOTTOM SLAB,
CINDER CONCRETE
BETWEEN STEEL BEAMS

FIG. 46.

half times as much as the same construction with the ends free. If this is true, it must follow that, for conditions between restrained ends and free ends, such as is pointed out for both top and bottom slabs, in actual practice, formulas and results between these two extremes will result.

In testing the restrained slabs, the authors, in all cases, used wire mesh reinforcement. Why this was done is not known, unless it was, as the authors state, that the wire mesh was furnished free. What little rod reinforcement was tested was used only in the free-end slabs.

Of course, the rods in the ideally constructed free-end slabs would appear poor when compared with the wire mesh reinforcement in the ideally constructed restrained slabs.

It seems strange that rods, so generally used in stone concrete slabs, were not used to some extent in the restrained slabs, thus affording a comparison of the two kinds of reinforcement.

It has been stated by one of the authors that the top slab construction, when brought down level with the top flanges of the beams,

Mr. Waite.

as shown in Fig. 45, will test higher than when the concrete and the reinforcement are both carried continuously over the beam, as in the tests. If it is true that when the reinforcement is carried over the beam loosely and left exposed it gives higher results than when it is brought over in a truly continuous manner, it would be evident that the continuous reinforcement over the beams has no influence on the carrying capacity. In that case any non-continuous kind of reinforcement will give equally good results. This, however, is denied by the authors when they state that their formula is "for slabs restrained between steel beams, with reinforcement thoroughly anchored, such as continuous mesh with proper cross-wires".

Therefore, there seems to be no escape from the fact that their formula for continuous reinforcement is not applicable to actual constructions.

It would seem as though data on the partly restrained construction, where one side of the slab is next to the walls, or next to the openings in floors, would have produced practical results. For some reason, only the two extreme types of slabs were tested, the rods being used on the free-end slabs. The result was that one type is two and one-half times as strong as the other.

The speaker would respectfully call attention to the experiments made at the University of Illinois, under the direction of Arthur N. Talbot, M. Am. Soc. C. E.,* in connection with the work of the Special Committee on Concrete and Reinforced Concrete. The purpose of the Committee was to gather data and suggest the proper manner of using concrete. These tests are used authoritatively in most recent treatises on reinforced concrete.

Inasmuch as rods, plain or deformed, are used almost exclusively for stone concrete (or any other concrete when in open competition), they were used also in these tests. After years of testing and after consideration on the part of the most talented engineers, the Committee, in its report,† recommends that for continuous slabs the bending moment be taken at $\frac{wl}{12}$ and that for simple slabs (with free ends) it should be taken at $\frac{wl}{8}$. In other words, continuous stone concrete slabs, which are easy to make truly continuous, as seen by Fig. 47 (as there is no steel beam to obstruct the reinforcement), are only given a capacity one and one-half times that of the free-end slabs.

REINFORCED STONE CONCRETE

Fig. 47.

* *Bulletins Nos. 4, 8, 12, 14, 28,* and *29,* University of Illinois Engineering Experiment Station, issued during 1906 to 1908.

† *Transactions,* Am. Soc. C. E., Vol. LXXVII, p. 385.

Mr. Waite.

This differs somewhat from the results obtained by the authors on the cinder concrete, a material which they assert is extremely variable. In other words, they conclude that if wire mesh reinforcement is used, the variable, cinder concrete, continuous slabs attain a value nearly twice as great as recommended by the Special Committee for the more uniform stone concrete.

The authors seemingly go outside the data of this paper to call attention to assumed shortcomings of all kinds of rod or bar reinforcement. They state* that $350\ 000\ 000\ \frac{da}{l^2}$ applies to slabs where continuous mesh with proper cross-wires is used; and that restrained slabs having longitudinal bars (of any kind) hooked over the beams are to be allowed but $250\ 000\ 000\ \frac{da}{l^2}$. That is, bars (of any kind) hooked over the steel beams, they assume, are only two-thirds as strong as wire mesh. If the authors consider the wire stronger in tension than the rods, then the most that could be allowed in good practice would be from 16 000 to 20 000, or four-fifths instead of two-thirds.

They also mention that rods (of any kind) are deficient, but that wire mesh with sufficient cross-wires was found sufficient—though they do not explain what quantity of cross-wires is needed.

The speaker can find no data in the paper justifying such conclusions. He can find no data relating to actual tests of rods hooked over the steel beams as compared with tests of continuous wire mesh.

The authors claim that four slabs failed, due to the slipping of the rods in concrete. On these few samples they would base broad conclusions, and these without satisfying the speaker that there was actual failure in the manner assumed.

The speaker wishes to call attention to tests made by Professor Talbot, especially to those on the bond stress of rods in various mixtures of stone concrete, reported in the *Bulletins* mentioned previously. After extensive testing he concludes† that failure by slipping is difficult to detect because it is often the result of some other cause. He concluded (in the same *Bulletin*) that there was no failure in his tests, due to slipping, in the use of mild steel plain bars. In *Bulletin No. 8* he gives as the bond stress of plain rods of mild steel 350 to 450 lb. per sq. in., and with deformed bars a result somewhat higher.

The speaker also begs to call attention to tests on the bond stress of both plain and deformed bars in 1:2:4 cinder concrete at Stevens Institute.‡

* *Proceedings*, Am. Soc. C. E., for February, 1915, p. 317.

† *Bulletin No. 4*, University of Illinois Engineering Experiment Station.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1787.

Mr. Waite.

It would seem to the speaker that the authors are not justified in making conclusions from the results of only these four samples. In any event, rods which are hooked over the steel beams, or bent up in any way, have more resistance than mere bond to connect them to the slab. Further, in some deformed bars the projections are such as to offer an almost continuous resistance, independent of the bond of the bar proper.

Once more attention is called to the statement of the authors:*

"In the case of mesh, the cross-wires give sufficient mechanical bond".

For those who are not familiar with the construction of the wire mesh referred to (not explained by the authors), the speaker wishes to state that the triangular wire mesh is fencing wire, and that the cross-wires are not mechanically fastened to the tension wires, but are simply twisted around them. In the case of the electrically welded wire, the cross-wires are mechanically fastened, but occur, usually, only once in about every foot of length of the tension wires. It is to be understood that this wire mesh was not originally intended to be used as reinforcement in concrete, but to keep hogs and other animals from getting out of pastures. Notwithstanding the apparent insufficiency of the mechanical means to insure proper bond in the wire mesh, as compared with the deformed rods (which are especially made for the purpose), the authors gave a vast preference to the wire mesh.

The strong claims for using wire mesh instead of loose bars are made because the tension members in the wire mesh are spaced uniformly apart by the cross-wires. The importance of correct spacing is made superior to the importance of a sufficient quantity of reinforcement. Some engineers think the quantity of reinforcement is the more important. As a matter of fact, when a sufficient number of rods are used, even if not accurately spaced, the concrete is ample to distribute across from one tension member to the other. Further, there are numerous spacing devices which compel the accurate spacing of straight rods.

In the case of stone concrete slabs, in which engineers specify a definite sectional area of tension steel, bars of some kind are always used, because there is no other form of reinforcement which can compete in price. The paradox in the competition of wire mesh, is not because of its cheapness per pound, but because of the lack of pounds which are used, on account of its supposed magical powers.

The speaker has no commercial interests which he is trying to preserve by the discussion of this paper. He wishes to see the truth brought out and the fittest material survive. The peculiar interest taken by him is prompted by his experiences for the last 15 years in connection with investigations of new building codes for New York

* *Proceedings*, Am. Soc. C. E., for February, 1915, p. 317.

City. He believes that nearly every one of the many proposed building codes have been instigated by interests which hoped to be benefited thereby. Mr. Waite.

If the conclusions of the authors are allowed to be misunderstood, the writer can see how the paper may be used as justification of the present system of using cinder concrete in New York City, and how the one kind of reinforcement may be specified in the coming new code, which is now being prepared, to the exclusion of all others, as was attempted in the codes of 1912 and of 1913.

If cinder concrete is elevated to a plane where engineers use it according to units of strength, as has been done with stone concrete, then each engineer should be allowed his discretion as to what kind of reinforcement he thinks best. Certainly, in the minds of most engineers, these tests have not proved that wire mesh is superior to the forms of reinforcement generally used. If any one is thus convinced, then it can only apply to the two conditions tested. These two conditions are not those of the weakest part of actual floors.

Until all the actual conditions encountered in practice have been tested, conclusions and recommendations may be erroneous, and may lead to dangerous results.

The speaker thinks that at present it is safer to rely on the recommendations of the Special Committee on Concrete and Reinforced Concrete for the proper formulas by which to design cinder concrete, using 30 for the ratio of moduli and 300 for the extreme fiber, as recommended by the authors, than to trust to new formulas based on limited tests.

The formulas of the Special Committee are simple in form, generally understood, and undoubtedly express the actual conditions in buildings more closely than any others.

Albert Oliver,* Esq.—It is undoubtedly a fact that the tests, demonstrated so ably by the authors, go a long way toward deciding, in the minds of both advocates and skeptics, that cinder concrete fireproofing should be considered a safe and proper construction for buildings of any class, except possibly those in which the most extreme conditions of loading and vibration obtain. However, there are many instances of the use of cinder concrete in much greater spans than those shown in the paper. Mr. Oliver.

About ten years ago, the Singer Manufacturing Company erected several buildings of great dimensions at St. Johns, Que., Canada. Mr. Phillip Sidney, the Engineer of the Company, decided that reinforced cinder concrete should be used for all floor and roof slabs. There were three spans across the buildings, two outer ones of 19 ft. 10 in. from the wall to the center of the supporting beam, and an interior span of 16 ft. 10 in. from center to center of the supporting steel

* New York City.

Mr. Oliver.

beams. The two outer spans rested on a 6-in. brick corbel at their edges, and were what is known as unrestrained slabs, that is, there was no continuity of the reinforcement beyond the outer edge of the slab. The reinforcement was 2 by 12-in. mesh, Nos. 3 and 10 Clinton electrically welded wire, to which was added ½-in. steel rods 12 in. apart. The concrete was a 1:2:4 mixture of cement, sand, and cinders, the outer slabs being 8 in. and the interior slab 7 in. in thickness.

As it was found impossible to get hard coal cinders, those from soft coal were obtained from the Canadian Pacific roundhouse at Farnham Station, and some were secured from Montreal. In no case were any hard coal cinders obtainable or used.

The speaker thought it was a crime to attempt such work, and so expressed himself. The engineer, however, had just completed a large quantity of work at South Bend, Ind., and had thorough confidence in the strength and other good qualities of cinder concrete. He felt himself well enough posted to be responsible, and did not court information on the matter. The work was done by the contractors with great care. The spans were very great, and the cinders were poor, but the cement and sand were of standard quality.

Rows of planers were placed on the 19 ft. 10-in. spans on each side of the buildings, the bays being 19 ft. 10 in. by 150 ft. On the 150-ft. stretch, however, the planers were set only far enough apart to permit of small hand-trucks being passed in for the removal of the planed cabinet work. The vibration, of course, is perceptible at all times when the machinery is in operation. In the other buildings there are great piles of castings and hardwood, and the speaker has always considered the load too great for the carrying capacity of these arches, yet, in all these years, not one crack or weakness of any nature whatsoever has developed in these floors.

The speaker, accompanied by R. D. Bradbury, Assoc. M. Am. Soc. C. E., visited these buildings about a year ago. On the day of this visit, a wheel pocket was being extended. To advance one of the machines into the room, it was necessary to cut a 10 by 36-in. slot into the cinder concrete arch. The Master Mechanic informed the speaker that it had taken two men 14 hours to cut out the slot. This offered a chance to observe the condition of the reinforcement. Samples were taken and are now in the possession of Mr. Bradbury and the speaker. It was found that the wire was in first-class condition, that corrosion had not set in, that the galvanizing was still on the wire, and that there was no indication whatsoever of any deterioration. The slot was placed between two bars, so that it was not possible to observe the condition of the black steel reinforcement.

The speaker believes that corrosion can only be prevented, especially where the reinforcement is not galvanized, by using a proper

quantity of water in the concrete, and if it is difficult always to get the correct quantity, he believes it would be better to use too much water than not enough. The best results can be obtained by having the concrete so soft that to tamp it would be to splash it. When concrete for floor and roof arches will stand tamping, it is certain proof that it contains insufficient water. In getting into position, the first resistance met by a soft concrete which will flow is the reinforcement, and as it is certain that the cinders are not the first to flow past or around the reinforcement, it must be the finer part of the mixture and, therefore, the water and cement part must form the skin coating around the reinforcement. If water and cement are placed thoroughly around the reinforcement, corrosion will be almost eliminated. Any one who wishes a demonstration of this fact may readily obtain it by encasing a piece of steel in concrete, as described. After the concrete sets, break it away from the steel and begin to shave into that part of it which came in contact with the steel and one gets cement. The sand will not be far back, but, nevertheless, it is back of the actual coating, which will be cement. Mr. Oliver.

The present use of cinder concrete for fire-proofing purposes is almost entirely the result of fire, water, and load tests on various systems, the term, system, being in most cases applied to the reinforcement used, for instance, the Roebling system, the Rapp system, the Clinton system, the Waite system, etc. Some tests have been very loosely conducted, but, in the poorest, there has not been, to the speaker's knowledge, a demonstration of the inability of cinder concrete to withstand the action of the fire test, as prescribed for New York City.

Undoubtedly, the best demonstration of the actual fire-proof qualities of cinder concrete is that given in the test house erected for Mr. Ira H. Woolson, at Norman and Monitor Streets, Long Island City. The successive fire and water tests to which its walls have been subjected prove beyond question the fact that a cinder concrete slab is less vulnerable to the action of fire and the application of water than any other building material now in use, not even excepting brick. The speaker has examined parts of the wall in this structure which have not been exposed to water, and has found that the grain of the lumber which formed the centering, when the walls were being poured, is still visible, thus proving that, where water is not applied to the heated surface, it is not possible to detect damage or deterioration in the concrete.

The old style of test load shown by the authors was that prescribed by the Bureau of Buildings, and that manufacturers and contractors made their tests accordingly was only to be expected. There is not much doubt that an arching effect occurred in many cases, but the speaker can show that in certain load tests made under Mr. Strehan's

Mr Oliver.

supervision, where the load was imposed on joists set on edge inside the line of the haunches, the joists being separated about 2 in. from each other, it was very difficult to trace an arching effect, and on these bearings loads were placed to such a height that Mr. Strehan asked for the discontinuance of the test, because it was regarded as dangerous to carry it further on account of the tendency of the load to slip and perhaps injure the workmen.

However, manufacturers and contractors may congratulate themselves that the new method of testing is now in use, and there does not seem to be the least reason for retracting the claims made heretofore that construction of this class will carry immense loads in a perfectly safe manner.

The speaker believes that the loading on third-points is the proper method, and is that used by J. R. Worcester and Rudolph P. Miller, Members, Am. Soc. C. E., Mr. Ira H. Woolson, Professor MacGregor, and the authors. Charles T. Main, M. Am. Soc. C. E., of Boston, Mass., A. W. French, M. Am. Soc. C. E., of the Worcester Polytechnic Institute, Messrs. Balcom and Darrow, and Warren and Wetmore, of New York City, all insist on center loads being applied, and tests made in both ways have failed to show any weakness in an ordinarily well constructed cinder concrete slab. Professor French insists on testing at the same time an outer and an inner arch, so that the effect on one in which the reinforcement is unrestrained may be compared with that on one having continuous reinforcement over the beams.

Fred W. Abbott, M. Am. Soc. C. E., lately conducted a load test on an isolated section of slab in the building being erected for Mr. P. A. B. Widener, of Philadelphia, Pa., Mr. Horace Trumbauer, Architect, and found, on a slab erected during the winter, a carrying capacity so far beyond the requirements of the Philadelphia Bureau of Buildings that he had every reason to conclude, with the concurrence of the officials of the Building Department, that reinforced cinder concrete formed a first-class construction.

There are matters affecting construction of this class which demand investigation by engineers. The matter of fire resistance has been adequately proved and may be set aside as an accomplished fact. The matter of the strength of a soft mass of 1:2:5, or better still of 1:2:4 cement, sand, and cinders, with the steel reinforcement exactly placed as to centers and held in place while the concrete is being deposited, may also be put aside as an accomplished fact; but what can be said as to the absolute certainty of these matters when loose members are used for reinforcing and when their placing is delegated to careless, low-priced, uninterested labor, and this is exactly the class of labor engaged in placing the vital agent in reinforced concrete, both cinder and stone. The speaker has photographs

which show the placing of a certain kind of loose members in the concrete of an important structure in New York City. No three members are equally spaced, and not a single member is on a line at right angles to the supporting flanges of the beam. The only straight pieces of steel running at right angles in the bays are the tie-rods. Mr. Oliver.

Now, the speaker asks, is this good practice, and does not such construction constitute a menace to life and property? He believes that the feature in this type of construction, which calls for the instant attention of the engineer, is not the fire-proof quality of cinder concrete—that has been proven—nor its load-carrying capacity when properly reinforced—that also has been proven—but the careless and improper spacing of reinforcement and the further abuse of actually leaving out many loose members where such reinforcement is specified; these are the particular points that call for drastic action by the Engineering Profession. The action must come from the engineer, whether he is employed by the city or in private practice. It must come from the Profession and not from the layman.

The speaker will offer to conduct a committee from the Society and prove every statement made as regards the placing of loose reinforcing members in concrete.

The authors have dealt with the matter of flaky soft substances in cinders. From experience, the speaker can say that from office buildings, hotels, gas and electric-light plants, the cinders gathered in New York City are of a high class. However, those obtained from certain congested points in the city, where more or less refuse is bound to be dumped, are not of good quality, and precaution should be taken at the building to reject any load of cinders showing more than an ordinary content of ashes or flaky materials, and any load wherein waste or refuse can be detected should be ordered off the premises.

When one considers that the area of cinder concrete floors and roofs already erected in New York City amounts to tens of millions of square feet, and that in no building where this construction has been used has a fire collapsed a single arch, and that not a serious break has occurred in any arch which has been in place for more than 30 days, it can be readily stated that to ask for the screening of the cinders, which form such an important part of this construction, would be to load up a first-class construction with an expense that would not be justified.

This matter has been fought out before the Code Committee of the Board of Aldermen. No more strenuous fight was ever conducted for and against any proposed change than that conducted by all interests concerned in the fire-proofing section of the proposed Code. During the past 10 years, the matter has been pretty well fought out, and it would seem that all are thoroughly satisfied that it should finally come up for disposition by Mr. Rudolph P. Miller.

Mr. Low. EMILE REED LOW,* ESQ. (by letter).—A test to determine the tensile strength of concrete made of granulated furnace slag, sand, and cement, as compared with cinder concrete, was made recently in connection with the construction of a pulp board mill, by the Beaver Companies, at Thorold, Ont. The specifications called for a 4-in. cinder concrete roof, mixed 1:2:4, and reinforced with No. 41 triangle mesh reinforcing wire, the concrete to be protected by an asphalted felt and gravel roofing. The final location of the new Welland Ship Canal had attracted numerous industries to this section, and these, together with the extensive work which the railroads were doing and the extensions being made to local factories, had placed a premium on cinders. In fact, during the middle of the summer, it looked as if it would be impossible to obtain cinders at any reasonable cost, and as pulverized furnace slag in large quantities could be secured cheaply, it was thought that slag might take the place of cinders.

A few years ago, the Carnegie Steel Company, of Pittsburgh, Pa., made a series of tests, extending over a period of 2 years, to determine the practicability of blast-furnace slag for use in concrete. The conclusions drawn from these tests showed that slag aggregate compared favorably with other materials, and that slag products have been used successfully in practice for several years. The concrete samples, however, were subjected to a compression test only. It was decided, therefore, to make a test for tensile strength and one that would follow more closely the special conditions.

Four concrete slabs were made up for testing, pulverized slag being used as the aggregate in two, and cinders in two. The slabs were made in wooden forms, 2 ft. 6 in. wide, 11 ft. long, and 4 in. high, and were reinforced with No. 41 triangle mesh reinforcing wire, manufactured by the American Steel and Wire Company. In all cases, the concrete was mixed in the proportion of 1:2:4 and was allowed to set for 7 days before being removed from the forms. The concrete was mixed in a "Wettlaufer" mixer to a consistency wet enough to run easily into the forms, and was not tamped as is usually the case in laying concrete roofs subjected to the great quantity of moisture present in paper and pulp mills.

Cement.—The cement was made by the Canada Cement Company, of Montreal, at the plant at Port Colburne, Ont., and two samples were tested, with the results shown in Table 18.

Sand.—The sand was secured from the pits of the Clifton Sand and Gravel Company, near Niagara Falls, Ont. It has a reddish brown color, and from a distance seems to contain a large quantity of clay. The color, however, is due mostly to the presence of iron oxide, and although a close examination shows the presence of some clay and loam, the sand has been used locally for several years, and has made excellent concrete.

* Buffalo, N. Y.

TABLE 18.

Mr. Low.

	Sample No. 1.	Sample No. 2.
Specific gravity	3.062	3.12
Water for normal consistency	24.20%	24.00%
Initial set	2 hours 15 min.	1 hour 15 min.
Final set	3 hours 30 min.	2 hours 30 min.
Fineness, 100-sieve	97.05%	97.1%
Fineness, 200-sieve	79.82%	83.7%
	Tensile strength, in pounds per square inch.	
1-day, neat	330	325
7-day, neat	575	575
7-day, 1 : 3 sand	125	125
28-day, neat	620	615
28-day, 1 : 3 sand	300	215

Slag.—The slag was secured from the Rogers-Brown Company, of Buffalo, N. Y. It is granulated by being run, in its molten state, into a pond of water. According to the statement made by this Company, the slag "averages about 40 to 50% lime, about 30 to 33⅓% silica, and about 16 to 18% alumina". The slag weighed 46 lb. per cu. ft., and ranged in size from ½ to 2½ in. The three elements contained in the slag are the essential elements of Portland cement, and, in a sample of this substance of good quality, average as follows: Lime, 62%; silica, 22%; alumina, 7.5 per cent. Although the slag aggregate contained the principal elements of Portland cement, and by pulverizing it, might be used as a cement, it would be low in strength and quick setting, due to the low percentage of lime and the high percentage of alumina. On the other hand, the high percentage of silica would tend to make a slow-setting cement which would not attain its full strength for a considerable time. By being granulated, however, rather than pulverized, it is not probable that the slag aggregate had any tendency to act as a cement.

Cinders.—The cinders were obtained from the Grand Trunk Railway, and were fair samples of the bituminous coal cinders made by the locomotives of this railway. Samples of the cinders averaged 45 lb. per cu. ft.

The slabs were tested by being weighed down with bags of cement which were placed symmetrically on them, beginning at the center and loading toward each end. The slabs were supported on 6 by 8-in. wooden beams, 10 ft. from center to center, this being the greatest center-to-center distance of the roof trusses in the building. The wooden beams were leveled and placed on solid foundations, so as to eliminate any chance of settlement.

The first test was made on one of the slag slabs when it was removed from the form, 7 days after the concrete had been mixed, and before

Mr. Low.

it had dried out thoroughly. Nevertheless, the slab held up very well, and, as shown in Table 19, was able to support a load of 125 lb. per sq. ft. before breaking. The fracture occurred about 12 in. from one end; the other three slabs which were tested broke very close to the center. One noticeable feature of the tests was that, under the lighter loads, the deflections of the 7-day slag slab were exactly the same as those of the 21-day cinder slab; and although the latter, before breaking, supported a much greater weight than any of the others, its deflections for the respective loads were greater.

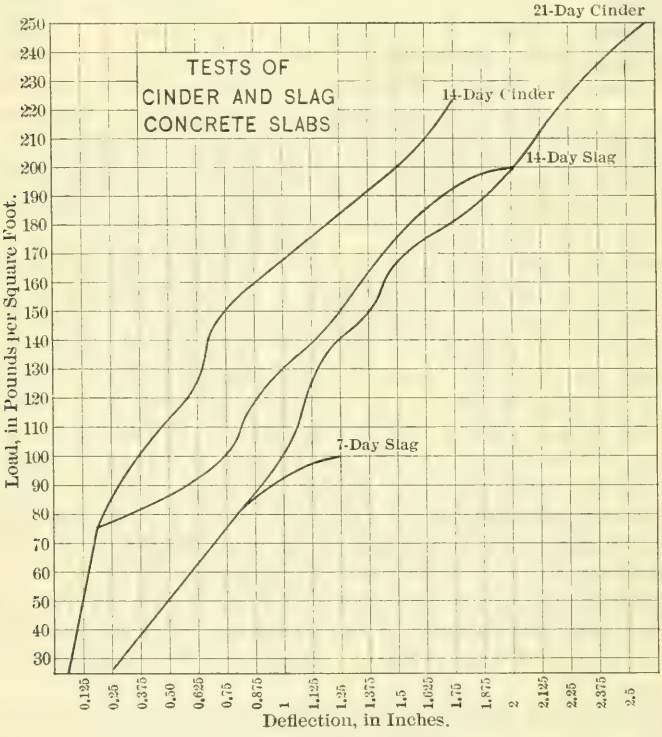


FIG. 48.

The second test was made on one of the cinder concrete slabs and the second slag slab, after they had set for 14 days, the first 7 days of which they remained in the forms, as stated previously. The tensile strength of these two compared very favorably, and although the ultimate breaking load of the cinder concrete was 25 lb. greater than that of the slag, the former showed signs of fracture with a load of 175 lb. per sq. ft., when a small crack appeared near the center of

the slab. The same feature was true of the 21-day cinder slab, which showed a slight fissure under a load of 225 lb. per sq. ft. On the other hand, the two slag slabs showed no signs of failure until immediately before their collapse; and yet their elasticity was nearly as great as that of the cinder concrete, as shown by the deflections of the different slabs, in Fig. 48. Mr. Low.

The tests were not extensive enough to provide data for general use, yet the results gave a very favorable comparison between cinder concrete and concrete with a slag aggregate. They also gave sufficient information to assure the engineers in charge of the work that sufficient strength would be secured for carrying the roof loads, whether cinder or slag aggregates were used. Before the work of laying the roof was begun, local conditions made it possible to secure cinders easily, and these were used in the concrete.

TABLE 19.—Deflections Under Various Loads.

Load, in pounds per square foot.	Slag, 7 days.	Slag, 14 days.	Cinder, 14 days.	Cinder, 21 days.	Remarks.
No load.	0	0	0	0	.....
25	0.25	0.0625	0.0625	0.25	.....
50	0.50	0.125	0.125	0.50	.....
75	0.75	0.1875	0.1875	0.75	.....
100	1.25	0.75	0.375	1.00	.....
125	Broke.	0.9375	0.625	1.125	.....
150	.....	1.25	0.75	1.375	.....
175	.....	1.50	1.125	1.625	Load of 175 lb. cracked, 14-day cinder.
200	.....	2.00	1.50	2.00	.....
210	.....	Broke.	.....	.....	.....
225	.....	.....	1.75	2.25	Load of 225 lb. cracked, 21-day cinder.
235	.....	.....	Broke.	.....	.....
250	.....	.....	.....	2.5625	.....
270	.....	.....	.....	Broke.	.....

Arthur H. Diamant,* Assoc. M. Am. Soc. C. E. (by letter).—The writer cannot agree with Messrs. Perrine and Strehan that the formulas derived from tests, as mentioned in the paper, apply to working conditions obtained in actual practice. Mr. Diamant.

Admitting that the cylinders were cast with cinder concrete mixed on the job, yet they were made by men familiar with constructing test cylinders. The test slabs also were made with cinders from the boiler room of Columbia University, carefully measured as to volume, and handled by men with more or less scientific training.

In actual practice, if the job is fairly large, the sub-contractor who delivers the cinders usually gets this material from a dozen different sources. The analysis of the cinders a short time after the test cylinders were cast, would probably show entirely different results from those in the tables.

* New York City.

Mr. Diamant.

The concrete is generally mixed in the cellar, where there is very little light. No attempt is made to measure the quantity of water, and a continuous mixer is ordinarily used. The labor is unskilled and foreign, and usually does not even understand the foreman. With this type of mixer, in many cases, the proportions of the aggregate are not always obtained. Although conditions such as these exist on construction jobs, the writer is of the opinion that no matter how carefully the theory of cinder concrete is worked up by the engineer, it does not agree with practice.

Mr. Woolson.

IRA H. WOOLSON,* ESQ.—This occasion should not be allowed to pass without a word in commendation of the excellent work which the authors have done. It is pioneer work in many ways; it has been performed with a great deal of conscientious care, and the results will make for great advantage in the use of this excellent material for fire-resistant purposes. The authors state that they are continuing the work, and undoubtedly their subsequent tests will add much to what they have learned in this preliminary investigation, and when the work is finally rounded out, and all the various systems of construction, different types of reinforcement, different methods of using the same type of reinforcement—after all that is completed—we will have something that will serve as a textbook which engineers can use to great advantage.

To those who have been through the experiences of the last ten years, in the hearings before the Board of Aldermen in New York City, in regard to the various types of fire-resistive material which should be accepted in Building Code regulations, this particular investigation is especially interesting. Those contests, as some know, were exceedingly hot. Days and weeks were spent in fighting over the various types of construction, and cinder concrete was usually the center around which the battle waged.

The speaker has been particularly interested in some of the results shown in this paper. The statement was freely and often made before the Board of Aldermen, that cinder concrete floors, when attacked by fire, would eventually burn out and drop of their own weight, because of the large quantity of unburnt coal which was contained in them. The speaker had occasion at that time to be in the midst of this squabble, and always contended that the quantity of coal found in the ordinary run of cinders had very little, if any, effect on its fire-resistive qualities. He made the statement at that time and was bitterly attacked for his one-sided opinion, that cinder concrete was the best fire-resistive material to be had; in other words, that its conductivity was the highest of any of the materials that were being used for floor construction at that time, and nothing has occurred since then to change that opinion.

* Cons. Engr., The National Board of Fire Underwriters, New York City.

Mr. Woolson.

Cinder concrete has its faults, and they are recognized; it has its weaknesses, in certain respects, but that it is a fire-resistive material, must be admitted. The authors refer to the circumstance that the speaker had dug coal out of the walls of the reinforced cinder concrete fire-test building, within about 1 in. of the inside surface. That is true, and is most remarkable. A few days ago the speaker repeated the inspection on that building, with the same result.

The authors state that the building has had a fire test of 28 hours; it has had more than that, at an average temperature of 1 700° for the whole time, and it is safe to say that it had a temperature of more than 2 000° for a considerable portion of that time. Pieces of bright unburnt coal could readily be found within 1 in. to 1½ in. of the surface of those walls, showing the remarkable resistance to heat conductivity possessed by the material.

The percentage of coal found by analysis in the cinders varies from about 12 to about 28%, and, considering the care with which the samples were collected, that is a fair representation of the quantity of coal which would be found in cinders used in New York City. The claim has often been made that coal should be excluded entirely from cinder concrete, and several of the proposed building codes which have been prepared for New York City have been drafted with that distinct requirement. These tests show that the requirement was entirely unjust, and that engineers must change their ideas on that matter.

In regard to the particular walls from which this coal was dug, it may be stated that the house was built for the speaker about seven years ago by a man who had never before built a reinforced concrete wall or structure of any kind. He was ordinarily intelligent, but had never had experience of that kind. The cinders were gathered from five different sources in Long Island City, and were not sifted; they were taken just as they came and made into a 1:2:5 mixture. The house was built without any supervision, other than that of the man who put it up, and it is giving splendid service in the work for which it was designed. It is good to-day for as many more tests as have already been made in it without any repairs except a little plastering where there has been local injury.

Another interesting thing in connection with that fire-test building is that the interior surfaces of the six chimneys which have taken up the products of combustion from all those fires, are to-day as smooth as they were when built. There is no pitting, and the little fins of concrete which formed between the boards of which the forms were constructed, stand there just as they did the day the chimneys were completed. This is another proof of the fire resistance of this material.

In regard to the question of design of cinder concrete floor arches, there will always be more or less difference of opinion. In time, it

Mr. Woolson.

will doubtless be put on some reasonably acceptable engineering basis. The work done by the authors is of high grade, and deserving of our best commendation.

Mr. Worcester.

J. R. WORCESTER,* M. AM. SOC. C. E. (by letter).—The information contained in this paper is valuable on account of the meagerness of data available with regard to this material. The results of the tests emphasize the wide variations which may be expected from cinders such as are commonly used in floor construction. There is little doubt that if a strictly vitreous clinker could be obtained, and all ash excluded, it would make a comparatively strong and reliable concrete; but the main object of using cinders would be lost, namely, the economy of the product. Cinders such as are readily obtainable will probably continue to include a large and widely varying proportion of fine, uncertain material, and the desideratum is a safe method of making use of the material commercially available.

In reference to this matter, it may be noted that the results of tests of screened and unscreened cinders reported in Appendix IV, which led to the conclusion that a loss of strength is to be expected from screening, are surprisingly at variance with what the writer would have predicted from a limited experience.

Other than this, the conclusions and recommendations of the paper seem to be in accord with previous investigations, and to be strengthened by the present study, though it is to be regretted that the authors see fit to give equal support to the empirical method of proportioning and that by the common theory of flexure. There does not seem to be any logical reason for adopting an empirical formula in which exponents are so ruthlessly manipulated and which must be hedged around with limits beyond which they cannot safely be extended, when the theoretical treatment is so simple, and, comparatively, so satisfactory.

In the theoretical method, it is doubtful whether it is wise to recommend for continuous slabs the moment coefficient of $\frac{1}{20}$ instead of $\frac{1}{12}$, even if the strength of these test slabs at failure does seem to warrant it. The objections are obvious. In the first place, it is known that the moment over the support exceeds $\frac{1}{20}$. Secondly, with imperfect continuity, which may be due to partial loading, flexible supports, or interruption of adjacent spans, the positive moment may actually exceed $\frac{1}{20}$. Then, again, the results of these tests may be somewhat misleading on account of arch action and partly restrained

* Boston, Mass.

supports. There seems to be ample reason for a conservative value for this coefficient, and it would be better—if we should be ultimately convinced of the safety of decreasing our "factor of ignorance"—to do it by modifying our unit stresses rather than the flexure coefficients. Mr. Worcester.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E.—In regard to the coal in cinder concrete, it may be suggested that, if air can be excluded, the carbon can be heated to a very high degree before any combustion occurs. Mr. Williams.

As to the methods of testing, where the sample is so large that a testing machine cannot be used, the speaker confronted that problem a number of years ago, and it was solved in a somewhat different manner from that described by the authors. As it may be useful in connection with this or some similar investigation, and might otherwise be overlooked, it will be briefly outlined.

The problem was to test a large girder of reinforced concrete composed of two beams and a deck. The deck was about 4 ft. wide. The span was a little more than 20 ft., and, in order to get the load on it, use was made of a series of pipes, about 3 in. in outside diameter and about 20 ft. long. The pipes were laid crosswise, one end projecting slightly more than the other beyond the edge of the girder, and the longer overhang was supported on spring balances hung from a scaffold or frame. The pipes were spaced carefully at equal distances, with short pieces of plank spanning them in pairs. The bags were piled on the planks in single piles, thereby preventing arching and distributing the load over the whole beam. The pipes, being able to roll and adjust themselves with the deflection, transmitted the load to the beam in a practically uniform manner. This method proved to be very successful, and may easily be applied by any one who is interested in making tests on a large scale.

Of course, the extra length of pipe toward the spring balances made it possible to obtain the necessary load without having the pile of bags very high, and, in that way, without permitting the bags to come in contact with each other as the beam deflected. The work was carried on at the University of Michigan, under the supervision of Charles J. Tilden, M. Am. Soc. C. E.

* Ann Arbor, Mich.

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PAPERS AND DISCUSSIONS

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THE ST. JOHN LEVEE AND DRAINAGE DISTRICT OF MISSOURI

Discussion.*

By Messrs. Vernon M. Eager and J. S. Spiker.

Mr. Eager.

Vernon M. Eager,† Assoc. M. Am. Soc. C. E. (by letter).—This paper is indeed one of interest and value to those engaged in such operations, and is welcomed in a field in which the demand for practical information so far exceeds the supply, as it does in this class of reclamation. Mr. Strohl is to be highly complimented on the able manner in which he has presented the subject.

It is to be regretted that a comprehensive map of the district treated was not included in the paper for the benefit of those less familiar with the conditions encountered. Such a map, showing the relative shapes, areas, and elevations of the different classes of lands to be treated, together with the locations of the proposed improvements, would be of much value in clarifying the minds of those unfamiliar with such relations.

In the development of the details of the P. Field's Bayou outlet, would not an automatic gate or pair of gates prove more efficient for the purpose of maintaining the low level of the interior water than the type adopted? The paper is not quite clear on this point, and it appears to the writer that the period of play of an automatic gate under the conditions here imposed would be longer, and in many instances more effective, than that of a hand-operated gate.

The writer notes with interest the apparent diminution of subsurface flow at Point 3 on the low-level ditch. Would this not indicate

* This discussion (of the paper by R. M. Strohl, Jun. Am. Soc. C. E., published in March, 1915, *Proceedings*, and presented at the meeting of April 7th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† South Bend, Wash.

Mr. Eager. the existence of a sub-stratum of sand or gravel or a sub-surface channel extending to, and sloping toward, the bed of the Mississippi River? If such a condition exists, the discharge capacity of such channel or channels would undoubtedly tend to increase with the flow, until such time as their effectiveness is impaired or destroyed by human agencies.

Mr. Spiker. J. S. Spiker,* M. Am. Soc. C. E. (by letter).—Much credit is due to Mr. Strohl, from engineers engaged in drainage and levee work, for the information contained in this paper. It is especially interesting to the writer because he has known the district, in a general way, since 1892, that is, prior to any drainage work in Southeast Missouri. His detailed knowledge of the work proposed does not justify any serious criticism of the plans as outlined in the paper, for, as a whole, the latter is worthy of commendation. The general information contained therein is systematically arranged, and the reason for an economical design is ably presented.

The paper states that the drainage district is divided into two parts, and that the assessments of benefits in the districts are based on different conditions. In one district the benefits are reckoned on the damages done by high waters passing over and washing the high land, the lots, and improvements in the City of New Madrid, and, in the other, on the prevention of overflow from bank water.

The assessments of benefits in the City of New Madrid are made tangible and definite by the method presented, but the justice and equality of the assessments, by such method, would not be substantiated by many of the Courts.

Some judges have held that the improvements on a piece of real estate and the purpose for which such improvements are used, bear the same relation to the real estate that personal property bears to the private individual; and the benefits derived by a method based on the present value of real estate and improvements, in drainage and levee proceedings, are open to criticism and revision by the Courts.

The substantial and impartial method of ascertaining the amount of benefits or damages to any piece of property or real estate, through and by any improvement, should be based on a reasonable and fair value before and after the improvement is made. If the value before is greater than the value after the improvement, the difference should be awarded as damages to the property, but if the value after is greater than the value before, the difference should be awarded as benefits.

In placing values on property or real estate before the improvement is made, one should be governed, commercially, by what the property

* Vincennes, Ind.

can be used for, and not by what it is used for, in its present condition; and the same method should be used in placing values after the improvement is made. Mr. Spiker.

The writer has no fear of contradiction, from those who have had experience in making assessments for drainage improvements, when he asserts that, in more than 75% of the individual pieces of property or real estate examined, the benefits are in reverse proportion to the value before the improvement; that is to say, the less the value before the improvement, the greater the proportionate benefits from such improvement.

The method of making the assessment of benefits on what is called "ridge land" is variable, resulting in values ranging from $5 to $30 per acre, depending largely on the elevation, location, and the frequency of overflow, which are the three essential elements to be considered. The variation of from $5 to $30 per acre seems excessive for land receiving complete protection from overflow.

It is presumed that, in a general way, the lower the land the higher the assessment per acre, was the method used in making the assessments. This, the writer regrets, was his first experience. Twenty-six years of practical experience and observation of leveed districts have caused him to change radically his method of making assessments. The agricultural and commercial advantages of the high land, over the low land, are so perceptible, after the improvement is made, that additional reasons are superfluous. The bottom land assessments are based on the benefits derived from the prevention, or partial prevention, of overflows from back-water.

It is presumed that in the St. John Levee District all head-waters have been cut out by the construction of former levees, and that all the lands in this District have contributed their proportional part toward their construction. This District is to pay for the extension of the present levee toward New Madrid for a distance of 17 miles, and for the construction of the low-level diversion ditch. Justice and equity (the writer might say, the drainage and levee law) should demand, of all the drainage districts emptying water into this district, an equitable proportion of the expenses for the construction of the diversion ditch, or that they should pay damages for overflowing the lower land by their surface water; but, under the present system, the high land can say to the low land, and to the Courts:

"Some time in the future, we are going to construct a high-level diversion ditch to take care of our water, and until we get ready to do this, we will use the low land as a dumping ground for our water, so that when we do get ready, you can help us pay for our ditch, as we can then prove that you will be benefited."

"Human laws, though designed to secure justice, are of necessity, imperfect, and hence what is strictly legal is at times far from being equitable and just." (Webster.)

Mr. Spiker.

The schedule of percentages is based on reasonable assumptions, namely, the elevation of the water at the origin and the duration of its extreme height.

The relative difference in elevation of the water at the origin and the extreme point of overflow will not be as great on the Mississippi as on smaller streams, owing to the duration of the extreme height of the flood-waters. On the Wabash River below Vincennes, Ind., the difference is about 1 ft. in 5 miles.

Both gravity and hydraulic reasons cause a marked difference between the Mississippi and Wabash Rivers, relative to back-water curves, on account of the fall and the duration of floods.

The schedule of percentages in connection with Table 12 and the peak of assessments is very interesting. The information is concise, comprehensive and tangible, and the writer is of the opinion that some changes might be made in the percentage, from the peak to the high land; but, on close investigation, this would only be a difference of opinion which might resolve itself into further complications.

All drainage and levee laws permit assessment of benefits to public highways and railroads, and, in some cases, to telephone and telegraph companies, but the latter is not so apparent, for railroads and highways, in many State laws, are named as essential factors of public utility, and if the proposed improvement does not benefit public highways, it causes the loss of one of the essential elements for the improvement.

It is not the writer's intention to criticize the matter presented, but rather to commend it, and extend to all engineers who are engaged in drainage work his congratulations, for they are doing work which is a co-ordinate of Creation, when properly correlated. It is hoped that in the future many more valuable papers on drainage may be presented before this Society.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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A METHOD OF DETERMINING STORM-WATER RUN-OFF

Discussion.*

By Messrs. George W. Fuller and Charles B. Buerger.†

Mr. Fuller.

George W. Fuller,‡ M. Am. Soc. C. E. (by letter).—In the consideration of storm-water problems, it is a fact that the engineer is required to exercise sound judgment in several particulars. This is true whether he attempts to determine the future run-off by the so-called "rational" method, or by the Buerger or any other formula. In no instance is a conclusion sounder than the premises on which it is predicated. In some cases, furthermore, the final decision ought not to be based on the ultimate maximum run-off, but should relate to the construction of drains and appurtenances which provide for all ordinary storm flows, beyond which flows the additional construction work required would not afford benefits in proportion to the cost involved.

In the final analysis, therefore, the degree of excellence attending the design of a storm-water drain is associated largely with the question of judgment, and, to that end, Mr. Buerger's paper and the discussions thereon are helpful, in that they afford a good review of prevailing practice.

On a number of occasions, the writer has been obliged to consider probable future storm flows in connection with requirements for pumping stations and settling tanks for storm overflows, both with and without sterilization of the flow to destroy objectionable bacteria from reaching a waterway from which a public water supply is obtained. For problems of that sort, especially, it is believed that the

* Discussion of the paper by Charles B. Buerger, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Author's closure.

‡ New York City.

Mr. Fuller. formula proposed by Mr. Buerger has some marked advantages of convenient application, giving adequate results within reasonable limits of accuracy.

Mr. Buerger. CHARLES B. BUERGER,* M. AM. SOC. C. E. (by letter).—It is rather curious that, in the voluminous discussion of this subject, so little reference has been made to the records of actual gaugings of sewer flows after storms. One might almost think that, after sewers were built, they did not actually carry off storm-water, and that the quantity they appeared to carry off was of the least importance.

Much has been said about the form of this formula, the desirability of its use, its inconsistencies and irregularities, the limits of its possible application, and the greater desirability and convenience of other methods of determining storm-water run-off. Whether or not the results shown by its use in any way correspond with what facts show, has been ignored, and also the question as to whether other methods of determining run-off give results corresponding with actual conditions.

This is certainly attacking the problem from the wrong direction. The question which should receive the most, if not the only, weight is what is shown by the comparison of known gaugings and the results obtained by this formula and by other methods. In the paper the writer gave the first place to such investigations, and made a detailed comparison of formula results with such gaugings as were then available.

Mr. Sherman makes some slight reference to this point.† He rather deprecates the making of such comparison with gaugings, because the storm flows which have been gauged are not the maximum flows for which the sewer is designed. It is true that the calculated run-off by formula from a rate of rainfall less than the maximum for which the sewer is designed will give results varying somewhat from those which would be obtained for a full sewer. The discrepancy, however, will not be very great, and the comparison is possibly as useful as if it were made for a sewer running full.

Mr. Sherman also comments:

"It must also be borne in mind that in design it is of the first importance that the designer form a reasonable judgment of the future conditions of the district under consideration, and adopt his coefficients and constants so that the drains may be adequate when the district has developed, rather than at the present moment, except in the rare case of the district which is already fully developed. It appears, however, that in the comparisons made by the author, his formulas gave figures less than the actual gaugings in a large percentage of the cases."

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* New York City.

† *Proceedings*, Am. Soc. C. E., for November, 1914, p. 2865.

Mr. Buerger.

Whatever the future conditions may be, and whatever influence they may have on the selection of constants and the design of storm-water sewers to meet such future conditions, none of these factors can in any way affect the comparison between gaugings and calculated run-off under present conditions, using the constants for such conditions. It is certainly proper to note the fact that gaugings show at times a greater run-off than that calculated for the conditions existing at the time. Any formula which would give uniformly higher figures than the actual run-off for the same conditions, having in mind the rather erratic results obtained from gaugings, would certainly not be dependable in any way as a guide to determine probable run-off under assumed conditions. Where, for instance, the run-off under the same conditions may vary from 10 to 20 cu. ft. per sec. with a certain rainfall, it is not a fair criticism to show that a formula gives a run-off of 15 cu. ft. per sec. and that actual gaugings at times give higher results.

Attention has been called to the fact that the paper did not include any reference to the gaugings given in the report* of the Committee on Run-Off from Sewered Areas, of the Boston Society of Civil Engineers. This was not an omission; the paper was submitted in April and the gaugings were not published until June. The idea, however, that all gaugings should be used as a basis of comparison is eminently proper, and the writer submits a comparison of these gaugings with the results obtained by the formula. In selecting gaugings for comparison, only those rainfalls have been taken in which the maximum rate exceeded 1 in. per hour. Table 24 gives the results of gaugings in six places compared with the formula calculation, and Table 25 gives in detail all the rainfall records reported for the Twelfth and Diamond Street Sewer, in Philadelphia, Pa.

Taken as a whole, the comparisons in Tables 24 and 25 show a reasonable corroboration between the calculated and the gauged results.

There seems to be generally a curious misconception as to the nature of the results obtained by the so-called "rational" method. The rational method, of itself, is not such a definite one that all men can and do agree on its use and on the results obtained by it. In its application, there will usually be as many results (differing widely from each other) as the number of men using it. One man may use this method in one way and get results 25% higher than those obtained by the formula; another man may use what he calls a rational method and get results 25% lower than the formula will give. Because of this fact, any comparison of results by formula with those from so-called rational methods of calculation will be convincing only to the particular man who makes the comparison, when that particular man is satisfied—as he usually is—that his own rational results are the only ones worth

* *Journal*, Boston Soc. of Civ. Engrs., June, 1914.

Mr. Buerger.

considering. Another man, however, will make computations in a different way, and will be just as well satisfied that the formula errs in the opposite direction by an equal amount.

TABLE 24.—Comparison of Sewer Gaugings with Results Obtained by Formula.

City, and drainage area.	Date.	A	L	K	S	m	C	q_B (Buerger).	q_G (Gauging).
Hartford, Conn., Franklin Avenue at South Street (p. 348).*	July 25th, 1900.	477	8 050	1.00	2	4	0.22	0.26	0.18
Hartford, Conn., Franklin Avenue at Bond Street (p. 349).	July 11th, 1901.	263.5	6 400	1.90	3	6	0.24	0.65	0.50
Pawtucket, R. I., Newell Avenue District (pp. 335 and 355).....	Oct. 20th, 1906.	146	4 800	0.70	9	7	0.25	0.30	0.302
Newton, Mass., Hyde Brook Drainage Area (pp. 335 and 356).	Sept. 4th, 1907.	350	7 500	0.70	20	23	0.24	0.33	0.61
Newton, Mass., Hyde Brook Drainage Area (pp. 335 and 356).	Aug. 7th, 1908.	350	7 500	0.70	20	23	0.24	0.63	0.71
Wilmington, Del., Shipley Run Drainage Area (pp. 334 and 356).	July 25th, 1908.	174	5 300	1.95	30	40	0.65	3.10	3.10

* Data from *Journal*, Boston Soc. Civ. Engrs., June, 1914, pp. 334-356.

A rather interesting application of these imaginary rational results is to be found in Mr. Sherman's discussion.* He points out that in five cases of his tabulation, the formula shows a deficiency of 100% where his rational method shows a flow from roof areas not connected. The formula shows a zero flow, because there is no area tributary to the sewers. Mr. Sherman seems to find it possible by his rational method to obtain a run-off from roof areas where no area whatever is tributary. To the writer, it would seem that whether the area in question is a roof, a pavement, or an uncultivated field, there must still be some area for rain to fall on before it can run off and show some flow in the sewer.

A few of the members discussing the paper have devoted some attention to the question of the origin and logical form of the formula. Mr. Horner derives a similar, almost identical, formula, on certain assumptions, and points out that the formula is not of itself a rational one. Mr. Horton calls for the derivation of the formula, suggesting that its value depends in some way on this derivation and on the formula being logical. Such features do not seem to be of any importance. The formula was presented as empirical, and should stand or fall according to the accuracy of the result obtained by it. How this formula may be derived, and how logical or illogical it may be in form, seems to be entirely beside the question. Although the formula was started in its present shape and some attempt was made

* *Proceedings*, Am. Soc. C. E., for November, 1914, p. 2868.

Mr. Buerger.

to formulate the existing logical relations, it was changed both in form and in constants in an attempt to make it correspond with such results of gaugings as have been made available. It was considered by the writer that logic of form should be sacrificed to accuracy of results. As it stands now, it is intended only as an empirical expression which should give results corresponding with reasonable accuracy to what will be found by the average of gaugings made.

TABLE 25.—COMPARISON OF GAUGINGS IN COMBINED SEWER AT TWELFTH AND DIAMOND STREETS, PHILADELPHIA, PA.*

$A = 1\,360$, $L = 10\,700$, $m = 14$, $S = 5.6$, $C = 0.65$, population $= 100\,000$.

Date.	K	P	q (Buerger).	q (Actual).	Ratio.
1903					
June 10th.....	1.32	2.30	1.45	0.49	2.96
June 10th.....	1.09	1.75	1.10	1.33	0.83
1906					
May 28th.....	1.04	1.75	1.10	1.02	1.08
June 16th.....	1.30	2.35	1.50	1.05	1.43
Aug. 2d.....	1.00	1.80	1.15	0.88	1.31
Aug. 21st.....	1.00	1.80	1.15	0.94	1.22
Aug. 24th.....	0.60	1.10	0.65	0.71	0.92
Oct. 5th.....	1.45	2.50	1.60	1.02	1.57
1907					
May 16th.....	0.67	1.25	0.75	0.87	0.86
July 18th.....	1.25	2.20	1.45	0.94	1.54
July 30th.....	0.95	1.70	1.05	0.89	1.18
Sept. 28th.....	0.58	1.00	0.58	0.53	1.09
1908					
May 22d.....	1.06	1.75	1.10	0.84	1.31
June 15th.....	0.46	0.90	0.52	0.90	0.58
July 25th.....	1.06	1.75	1.10	0.96	1.15
Aug. 25th.....	0.75	1.35	0.80	0.72	1.11
Aug. 26th.....	0.62	1.25	0.75	0.70	1.07
1909					
Aug. 16th.....	0.52	0.95	0.55	0.56	0.98
1910					
Aug. 8th.....	0.60	1.10	0.65	0.61	1.07
Aug. 19th.....	0.97	1.80	1.15	0.94	1.22
Sept. 2d.....	0.65	1.25	0.75	0.41	1.83
Sept. 6th.....	1.08	1.75	1.10	1.06	1.04
Oct. 19th-20th.	0.64	1.25	0.75	0.59	1.27
1911					
June 3d.....	....	....	....	....	....
July 17th.....	1.00	1.80	1.15	1.09	1.05
Aug. 15th.....	0.56	0.95	0.55	0.88	0.63
Aug. 30th.....	0.69	1.30	0.80	0.88	0.91
Sept. 11th.....	0.49	0.92	0.54	0.46	1.17
Nov. 6th.....	0.45	0.92	0.54	0.51	1.06
1912					
Feb. 26th.....	0.95	1.80	1.15	1.01	1.14
Mar. 12th	0.75	1.35	0.80	0.81	0.99
Mar. 15th.....	0.95	1.80	1.15	0.98	1.17
Nov. 1st.....	0.34	0.55	0.30	0.59	0.51
Average ratio (excluding first item).....					1.10

* Data from *Journal*, Boston Soc. of Civ. Engrs., June, 1914, p. 357, and by letter from George S. Webster, M. Am. Soc. C. E.

Mr. Buerger.

Mr. Weston E. Fuller deduces, as a result of a comparison of the effect of the various factors, that "the relations existing between the factors, however, are not the true ones", and suggests that the formula is not unlimited in its usefulness, but that "for a more limited use, perhaps covering all but the largest of sewer areas, the formula may give results approximately correct for average conditions". This is, in fact, the only range for which it was designed. It is intended to give reasonable results for ordinary sewer areas, and is not to be applied to the run-off from large water-sheds. For such service, it is hardly possible, in the judgment of the writer, for any formula whatever to give reasonable results, and the only guides at all worthy of credence are records of actual run-off for the particular water-shed, considered in connection with the quantity of rainfall and the probability of increased or decreased rainfall in the future.

Mr. Grunsky points out* that the relation used for the reduction of rainfall to a basis of 1-hour periods is not suitable for use for such places as San Francisco and Kansas City, where the rainfall follows a different law and the total quantity of rain increases much more rapidly with the time than is obtained by the assumed relationship. This is a fair criticism, but the effect is by no means as marked as is suggested in Table 19. Within ordinary ranges, and in ordinary sewer districts from which storm-water is to be removed, a period of 2 hours for concentration may be considered practically the maximum. For such a limited period, the error of the assumed formula is not excessive. It is obvious, of course, that any formula intended for general application must be based on some median rainfall curve, and that those particular localities which have extremes—and San Francisco is one of these localities—will show the most marked departures from this median line. For such localities, the error of using a formula will be somewhat greater than that found in average practice. Whether such error will be excessively great depends on a number of local conditions, and these should be considered in selecting the method of determining the storm-water run-off.

Several references are made to the fact that the constant coefficient of run-off represented by C is not defined in the usual way. The writer defines this coefficient, C, as "the ratio between the run-off and the total rainfall",† the more common definition being the ratio of the maximum rate of run-off to the average rate of rainfall. This change was made intentionally by the writer, and this was pointed out when he stated that "the coefficient, C, probably does not, in all cases, represent the identical conception". In selecting a value of C for use in the formula, proper consideration should be given to this

* *Proceedings*, Am. Soc. C. E., for November, 1914, p. 2874.

† *Proceedings*, Am. Soc. C. E., for August, 1914, p. 1754.

definition, and the curve for C depending on the tabulation submitted by the writer is based on this revised definition. Mr. Buerger.

Some exception has been taken to the writer's statement that the formula permits of work which is largely mechanical in its nature and does not require the exercise of any high degree of judgment in its application. A little examination, however, will show very plainly that this statement is a moderate expression of the facts.

Taking the various elements of the formula, it will be found that L, the length of the sewer, is always determined by topographical considerations at the first glance, and does not require much thought. In addition, although the path of the sewer may be changed somewhat, the total length will not differ materially, whatever path be assumed.

C is the run-off coefficient to be determined for some future date when the district has increased in population and the run-off is greater than at the present time. The population diagram, Fig. 2, gives this coefficient, C, in terms of the future population, and thus this value requires no further thought.

The area of the district in question appears on the map, and does not need the exercise of any judgment.

The slope of the sewer is also a relatively simple matter; for all practical purposes, it is quite sufficient to take the difference of elevation between the highest point and the point under consideration, and divide this by the total length of the sewer, in thousands of feet, to get the slope per thousand. Local variations from this slope will not affect the result to such an extent as to warrant any investigation.

The value of the average slope to the sewer inlet is a little more difficult of determination; it requires the taking of the difference of elevation of the sewer inlet and the high point between sewer inlets at a number of places within the area in question; the average of these results is the value of m. It may take a certain amount of labor to determine this average slope, but the work is practically all mechanical.

The last factor of the formula, the value of K, can readily be derived on the assumption of the interval between full charging of the sewer and the rainfall curve, for the district in question, or for a district of somewhat similar characteristics.

A great deal of consideration has been given, and quite properly, to the question of the facility of use of this new formula and of its relative ease as compared with other methods. This, of course, is all a matter of experience in a particular use. It is natural that one unfamiliar with it would find its use cumbersome and somewhat difficult. The writer's experience suggests, however, that a reasonable amount of practice in determining values by the curves on a relatively large scale permits of very rapid and accurate work. The diagrams printed with the paper are, of course, too small to be suitable for any

Mr. Buerger.

readings at all. The writer has found that an assistant of ordinary intelligence is easily able to work several times as rapidly with the help of suitable diagrams for this formula as with any of the ordinary variations of the rational method. What is even more marked, however, is the fact that the work is much more likely to be accurate, and is much more trustworthy.

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PAPERS AND DISCUSSIONS

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NOMOGRAPHIC SOLUTIONS FOR FORMULAS OF VARIOUS TYPES

Discussion.*

By Messrs. W. M. Eliot and F. W. Green.

Mr. Eliot.

W. M. Eliot,† Jun. Am. Soc. C. E. (by letter).—The author has opened a new field to those engineers who hitherto have been unacquainted with this method of platting. As a general rule, any formula can be solved with nomographs by the elimination of one variable at a time. In an equation with several independent variables, however, it happens, frequently, that results are wanted for a few values only of one or more of them. The solution of the formula by using a nomograph can be made much simpler in this case by platting the few values required on the same graph.

As an illustration, in a recent issue of a technical publication‡ there appeared the following formula for the approximate number of rivets required in an eccentric connection (Fig. 36):

$$n = \sqrt{\frac{K\,(6\,L + m\,d)}{m\,p}}.$$

n = number of horizontal rows of rivets;

m = number of vertical rows of rivets;

p = vertical rivet spacing;

d = horizontal rivet spacing;

L = eccentricity of load, W;

K = ratio of load to the allowable rivet stress $= \frac{W}{S_R}$.

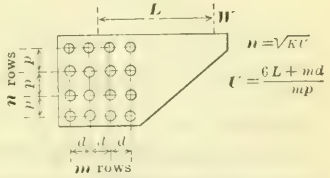

Fig. 36.

* Discussion of the paper by R. C. Strachan, M. Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† Dallas, Tex.

‡ *Engineering News*, October 29th, 1914, p. 868.

Mr. Eliot.

For ordinary detailing, values of $m = 1$, 2, 3, or 4, and $p = 2\frac{1}{2}$ or 3 in., will be sufficient to cover most cases.

The formula will be divided into two parts:

$$U = \frac{6\,L + m\,d}{m\,p}, \text{ and } n = \sqrt{K\,U}.$$

A nomograph will be made first for the solution of U, and then one for n.

$$U = \frac{6\,L + 2\,d}{2 \times 2\frac{1}{2}}, \text{ when } m = 2, \text{ and } p = 2\tfrac{1}{2}$$

$$\tfrac{5}{8}\,U = \tfrac{3}{4}\,L + \tfrac{1}{4}\,d.$$

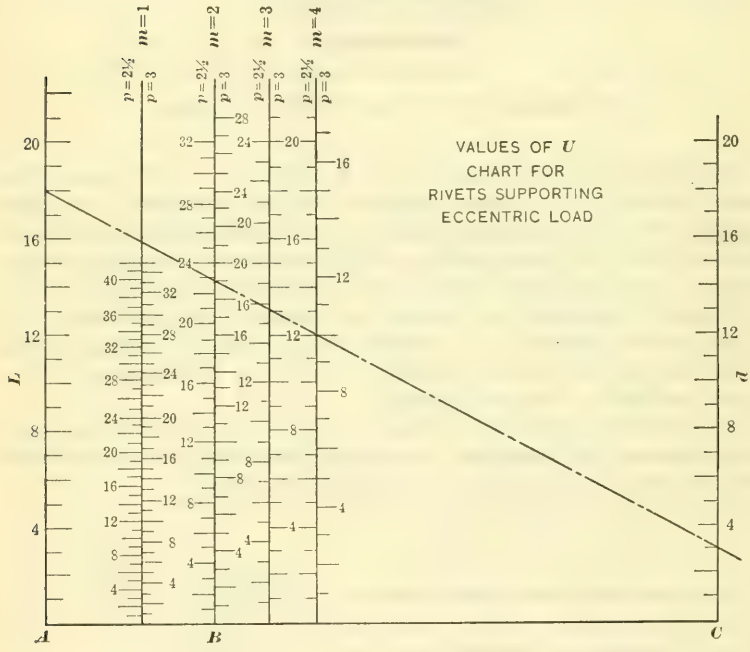

FIG. 37.

On the base line (Fig. 37), $AB = \frac{1}{4}\,AC$. The scale for $m = 2$, $p = 2\frac{1}{2}$, is five-eighths of that used for L and d. Other values of U are similarly platted.

Scales U, K, and n, are platted logarithmically (Fig. 38), lines for U and K being equidistant from n.

Given $K = 9\frac{1}{2}$, $L = 18$ in., $m = 4$, $p = 3$, and $d = 3$, to find n. On Fig. 37, place a straight-edge on 18 in. (L) at the left and on 3 in. (d) at the right; read 10.0 (U) on the line, $m = 4$, right side $p = 3$ in. On Fig. 38, place a straight-edge on 10.0 (U) at the left, and on $9\frac{1}{2}$ (K) at

the right; read 9.7 (n). Use four vertical rows of rivets, ten in each row. By this method, an expression containing six variables is solved for certain values with two nomographs. Mr. Eliot.

One of the main advantages of the nomograph is the few lines required to represent a formula. On this account, several results may often be shown on the same plat. The stresses in a roof truss (Fig. 39) of any slope and panel loading may be ascertained by the same general method as that used by the author for stresses in pins. As it is impracticable to use a separate graph for each member, they have been combined as shown in Fig. 40, two scales being used to give greater accuracy in the results. The stress in m 1 for a panel load of 2 000 lb., bevel of top chord 6 in 12, is 14 000 lb.

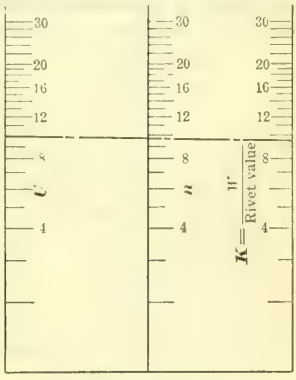

FIG. 38.

Other simple types of trusses have been similarly platted by the writer, but are not reproduced here, as the method is the same for all. By the use of these graphs, and diagrams of angles platted for strength in compression for different lengths, and for tension values with the proper size and number of holes deducted, it takes only a few minutes to ascertain the make-up of a roof truss for an ordinary mill building. An excess vertical load is assumed to take care of wind stresses, but this need not be the same for all members.

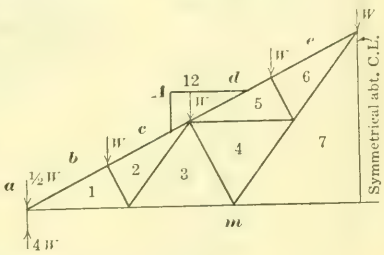

FIG. 39.

The writer does not believe the author's chart for reinforced concrete beams will be used to any extent by engineers, as the diagrams* by Arthur W. French, M. Am. Soc. C. E., cover the same ground in a simple manner, and are now in general use. The two nomographs which follow, though only for stresses of 650 and 16 000 lb. per sq. in. on the concrete and steel, respectively, give more direct results than the one proposed by the author. The diagram, Fig. 41, gives the size of beam and area of steel required to resist a given moment, when the concrete and steel are each stressed to the working limit. For the case shown, a moment of 428 000 in-lb. requires a beam 10 by 20 in. (effective depth), and an area of steel = 1.54 sq. in.

* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 360.

Mr. Eliot.

Often the size of beam is fixed, and it is desired to know the quantity of steel required for a given resisting moment. This is given by the nomograph, Fig. 42; the resisting moment is determined

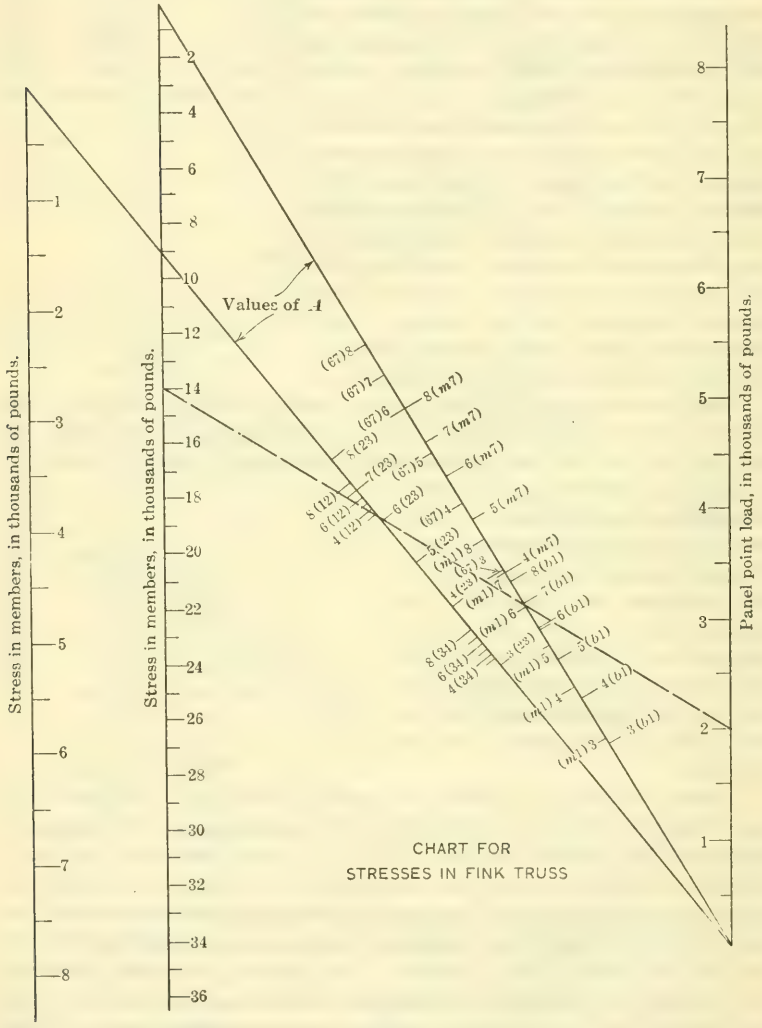
Stress in members, in thousands of pounds.
Stress in members, in thousands of pounds.
Values of A
Panel point load, in thousands of pounds.
CHART FOR
STRESSES IN FINK TRUSS

FIG. 40.

by the steel being stressed to the working limit for values of p less than 0.0077, and by the concrete for values of p greater than 0.0077. Given a beam, 10 by 20 in. (effective depth) to resist a moment of

350 000 in-lb.: Place a straight-edge on the moment 35 000 lb. (for 1 in. in width) and on $d = 20$; read $p = 0.0062$. Area of steel required $= p\ b\ d = 1.24$ sq. in. One designing many beams can well afford to make similar nomographs for the stresses he uses, as little time is required. Mr. Eliot.

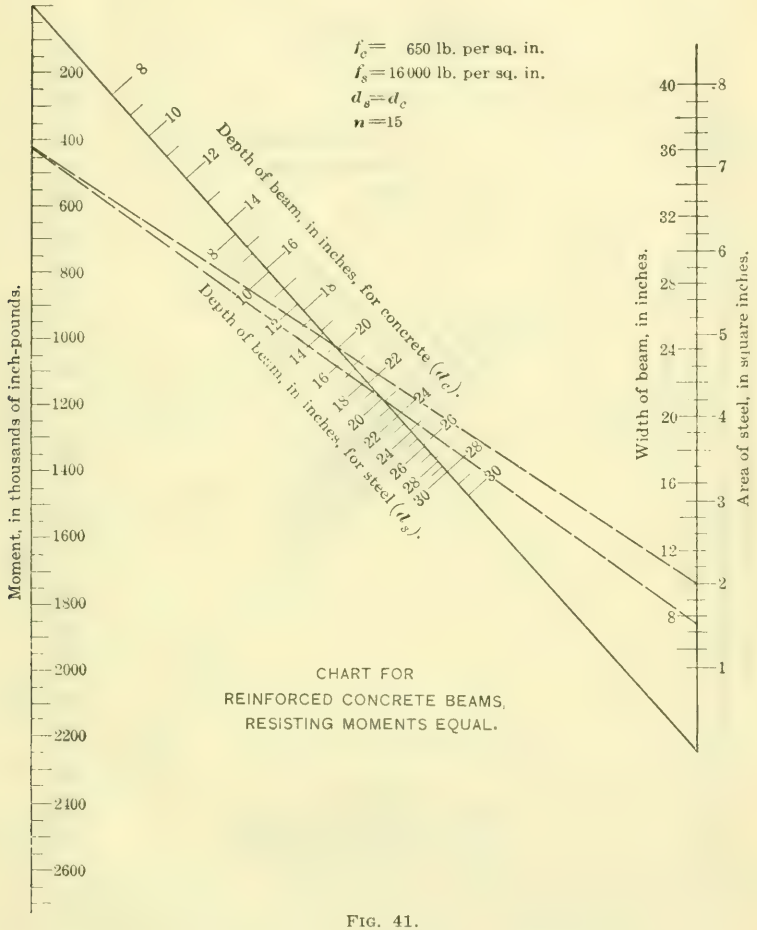
f'_c = 650 lb. per sq. in.
f_s = 16 000 lb. per sq. in.
$d_s = d_c$
n = 15
Depth of beam, in inches, for concrete (d_c).
Depth of beam, in inches, for steel (d_s).
Moment, in thousands of inch-pounds.
Width of beam, in inches.
Area of steel, in square inches.
CHART FOR
REINFORCED CONCRETE BEAMS,
RESISTING MOMENTS EQUAL.

FIG. 41.

The diagram, Fig. 43, gives the dimensions of a yellow pine beam directly, when the moment is known. If the total uniform load and span are given, two isopleths are required to ascertain the size of beam. In the example shown, a load of 19 000 lb. on a span of 20 ft.

Mr. Eliot.

requires a beam 10 by 16 in. (actual size 9½ by 15½ in.). The moment itself need not be read.

The pitch of rivets connecting the web and flange of a plate girder (see Fig. 44) is given approximately by the formula:

$$p = \frac{R\,h}{V},$$

$$\text{or, } \frac{10\,p}{h} = \frac{10\,R}{V} = Q.$$

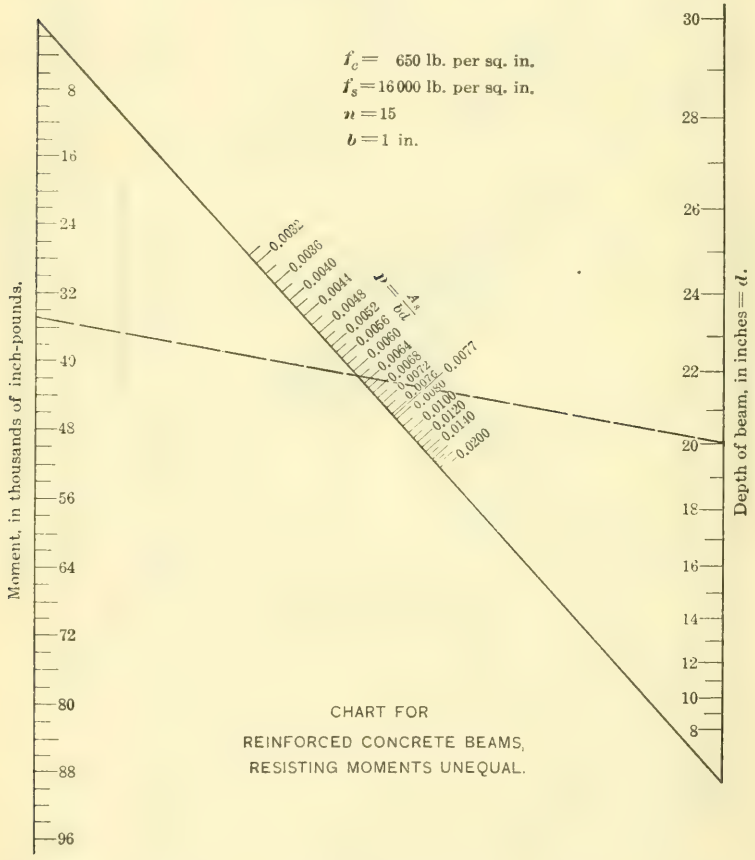

FIG. 42.

The isopleths joining p and h and R and V, intersect on the diagonal line for Q connecting the ends of the scales. Thus the pitch is 2.8 in. for a vertical shear of 200 000 lb. on a girder having an effective depth of 70 in., with a rivet value of 8 000 lb. As this formula

gives a pitch that is a little low, correction can be made to some extent by using the total for the effective depth of the girder. As the values of Q are not platted, nomographs for equations of this type are easily made. Mr. Eliot.

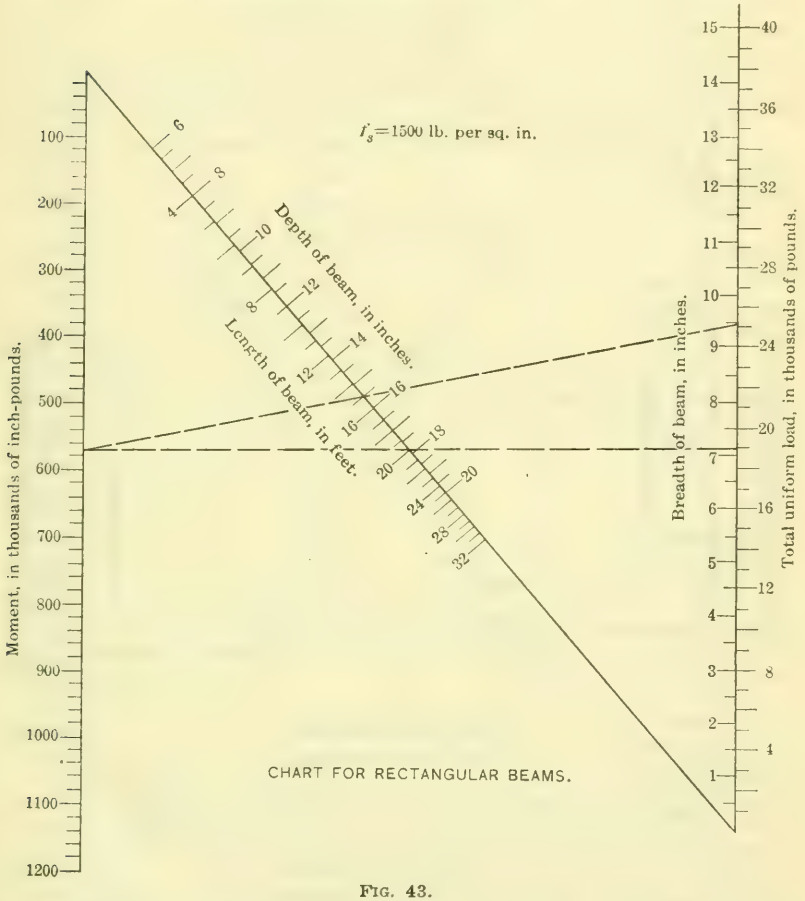
f_s=1500 lb. per sq. in.
Depth of beam, in inches.
Length of beam, in feet.
Moment, in thousands of inch-pounds.
Breadth of beam, in inches.
Total uniform load, in thousands of pounds.
CHART FOR RECTANGULAR BEAMS.

FIG. 43.

This method can be applied in the same way to more complex formulas, such as that for the deflection of beams (Fig. 45).

$$\Delta = \frac{5\ W\ L^3}{384\ E\ I},$$

$$\text{or, } \frac{384 \times 58\,000\ \Delta}{L^3} = \frac{W}{100\ I} = Q.$$

Mr. Eliot.

when $E = 29\,000\,000$ for steel;
Δ = deflection;
L = span;
W = total uniform load;
I = moment of inertia.

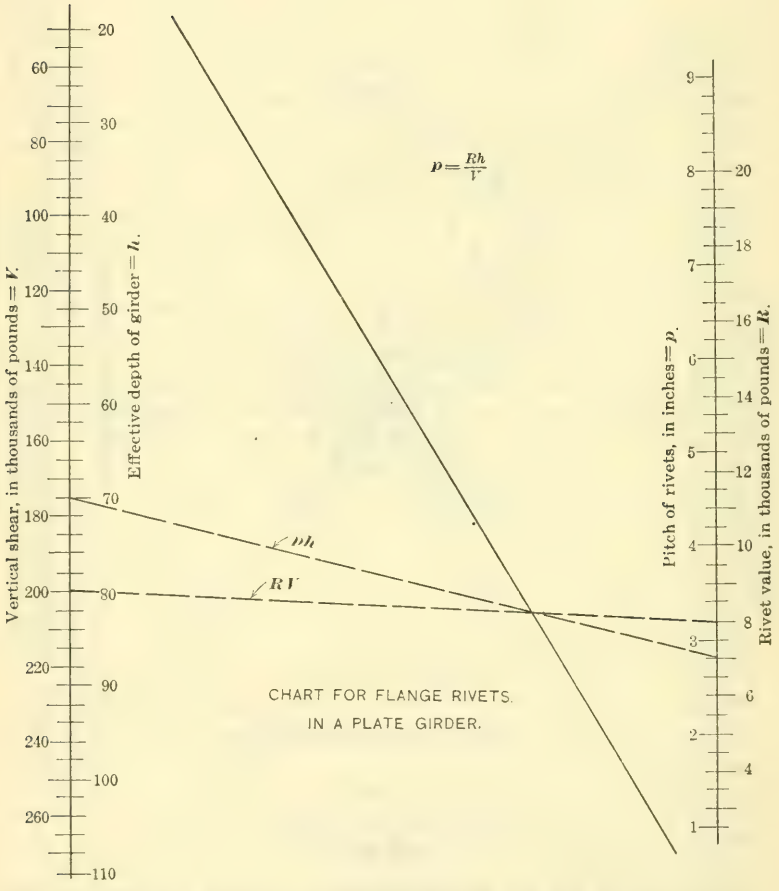
Vertical shear, in thousands of pounds = V.
Effective depth of girder = h.
$p = \frac{Rh}{V}$
Pitch of rivets, in inches = p.
Rivet value, in thousands of pounds = R.
ph
RV
CHART FOR FLANGE RIVETS
IN A PLATE GIRDER.

FIG. 44.

The cubes of L are used, and the beams are written at their respective moments of inertia when the members are platted. The deflections for a concentrated center load are similarly shown. A 20-in. I-beam of 65 lb. per ft., with a total uniform load of 50 000 lb. on a span of 25 ft., has a deflection of 0.52 in.

Mr. Eliot.

The writer, in presenting these nomographic solutions for formulas used in structural engineering, has not given the details of construction in each case, as the method will be apparent to any one who has read the paper. The alignment diagram is very useful in this field, even though many of the formulas are readily solved by

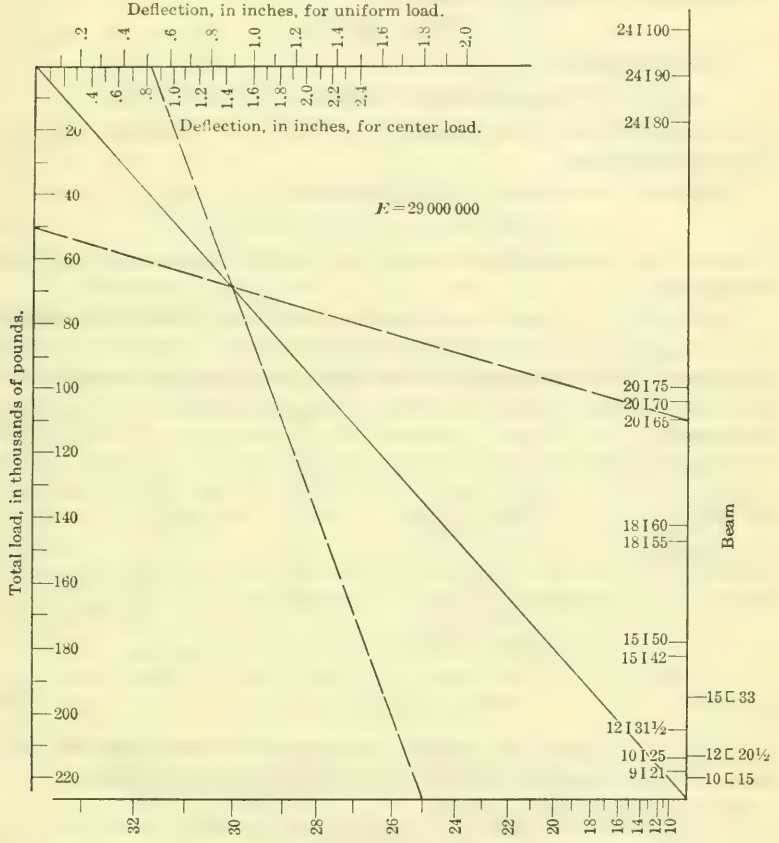

CHART FOR DEFLECTION OF STEEL BEAMS.

FIG. 45.

direct computation. It would seem that this method would be of greatest value, however, in hydraulic and experimental work, where fractional exponents frequently occur.

The author has ably presented this method of solving formulas. The system has heretofore been used to some extent, but its principles have not been generally understood.

Mr. Green. F. W. GREEN,* ASSOC. M. AM. SOC. C. E. (by letter).—As the author's presentation of this subject seems to the writer to leave rather too much for the reader to demonstrate for himself, the following discussion will be addressed to clearness of presentation rather than to the addition of anything new.

Assume an expression of the form,

$$Ax^p = By^q\ Cz^r.$$

Using logarithms,

$$\log.\ A + p \log.\ x = \log.\ B + q \log.\ y + r \log.\ z + \log.\ C,$$

in which A, B, and C are constants, x, y, and z are variables, and p, q, and r are exponents.

Dropping the constants for the time being,

$$p \log.\ x = q \log.\ y + r \log.\ z \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

which will be considered as the equation for which a nomograph is to be constructed.

In Fig. 46, y, x, and z are three parallel lines the scales of which are arranged so as to satisfy the required equation. The line, $A\,B$, is an isopleth. The arrangement of the three parallel scales in such a way as to solve such an equation is called a nomograph.

From geometry $fk : a :: z : a + b$,

whence $$fk = \frac{a}{a+b}\,z,$$

and, similarly, $kl : y :: b : a + b$,

whence, $$kl = \frac{b}{a+b}\,y.$$

As $fk + kl = x$,

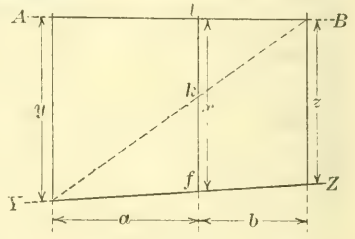

FIG. 46.

therefore, $$x = \frac{a}{a+b}\,z + \frac{b}{a+b}\,y \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Equation (2) shows the relation of the scales of values for the variables, which is required in order to satisfy the given equation.

The expression assumed was,

$$Ax^p = By^q\ (Cz^r,$$

whereas Equation (2), in the right-hand member, involves addition instead of multiplication. Therefore, logarithms must be used in order to make Equation (2) applicable, thus,

$$\log.\ A + p \log.\ x = \log.\ B + q \log.\ y + \log.\ C + r \log.\ z,$$

which, after dropping the constants, becomes,

$$p \log.\ x = q \log.\ y + r \log.\ z$$

which is Equation (1).

* Stamps, Ark.

Comparing Equations (1) and (2) shows that it is necessary to devise a method of converting the coefficients of z and y, respectively, in Equation (2) to those in Equation (1). Therefore, a modulus is used, thus: Mr. Green.

$$\frac{a}{a+b} \log. z\, m_1 = r \log. z,$$

whence,
$$m_1 = r \frac{a+b}{a} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\frac{b}{a+b} \log. y\, m_2 = q \log. y$$

whence,
$$m_2 = q \frac{a+b}{b} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Dividing Equation (3) by Equation (4),

$$\frac{m_1}{m_2} = \frac{r\,(a+b)}{a} \div \frac{q\,(a+b)}{b} = \frac{b\,r}{a\,q} \ldots\ldots\ldots\ldots (5)$$

By inspection of Equation (5) it is seen that m_1 and m_2 vary as a and b, respectively; or

$$a\,q\,m_1 = b\,r\,m_2.$$

Particular attention is called to this, which is the fundamental principle of the nomograph. The condition, $a\ q\ m_1 = b\ r\ m_2$, must be satisfied in every case, or any assumed isopleth fails to solve the given equation. If any changes occur, either in a or b, the corresponding changes must be made in the other factors. Until the reader has a clear conception of this he cannot make further satisfactory progress.

To illustrate this principle, one example will be worked out in detail. The author refers to a modification of the Williams-Hazen formula as

$$v = 0.0796\, d^{0.63}\, h^{0.54}.$$

For convenience in applying the formula, this will be written,

$$x = 0.0796\, y^{0.63}\, z^{0.54} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Using logarithms,

$$\log. x = \log. 0.0796 + 0.63 \log. y + 0.54 \log. z$$
$$\log. x - \log. 0.0796 = 0.63 \log. y + 0.54 \log. z.$$

a and b may be assumed as having any arbitrary value whatever. In this case, the exponents will be used for such arbitrary values, and it will be assumed that $a = 63$ and $b = 54$.

Then, from Equation (2),

$$\log. x - \log. 0.0796 = \frac{63}{117} \log. z + \frac{54}{117} \log. y.$$

Substituting in Equation (5) and solving for $\frac{m_1}{m_2}$,

$$\frac{m_1}{m_2} = \frac{b\ r}{a\ q} = \frac{0.54 \times 54}{0.63 \times 63} = \frac{29.16}{39.69} = \frac{1}{1.361}.$$

Mr. Green.

Draw three parallel lines for the x, y, and z scales, respectively, with the distance from y to $x = \frac{63}{117}$ of that from y to z. If the highest values desired are for $y = 60$, and $z = 100$, an isopleth may be drawn across these scales near the top (as the line, $A\,B$, in Fig. 46), and the intersection with y marked 60 and that with z marked 100. Substituting these values in Equation (6) and solving for x, gives $x = 12.62$, which value is marked on the x scale, similarly.

Next make a table (Table 1) in order to obtain values for plotting the various scales.

TABLE 1.

y (1)	log. y (2)	$m_2 \log y = 1.361 \log. y$ (3)	z (4)	log. z (5)	$m_1 \log. z = 1 \log. z$ (6)	x (7)	log. x (8)
60	1.778	2.420	100	2.000	2.000	12.62	1.101
50	1.699	2.312	90	1.954	1.954	12.00	1.079
40	1.602	2.180	80	1.903	1.903	11.00	1 041
30	1.477	2.008	70	1.845	1.845	10.00	1.000
20	1.301	1.770	60	1.778	1.778	9.00	0.954
10	1.000	1.361	50	1.699	1.699	etc.	etc.
9	0.954	1.298	40	1.602	1.602	.....	.....
8	0.903	1.228	30	1.477	1.477	.....	.....
7	0.845	1.150	20	1.301	1.301	.....	.....
6	0.778	1.058	10	1.000	1.000	.....	.....
5	0.699	0.952	9	0.954	0.954	.....	.....
4	0.602	0.820	etc.	etc.	etc.	.....	.....
3	0.477	0.649	.....	.....	.....	.....	.....
2	0.301	0.410	.....	.....	.....	.....	.....
1	0.000	0.000	1	0.000	0.000	1.00	0.000

Using any convenient scale (as the 20th) plot the values of y from the items in Column (3), and, similarly, with the same scale, plot the values of z from the items in Column (6).

Using the same scale, the values of log. x — log. 0.0796 are plotted as follows:

$$\text{log. } 12.62 = 1.101$$

$$\text{log. } 0.0796 = 2.901 = -2 + 0.901 = -1.099$$

$$\text{log. } 12.62 - \text{log. } 0.0796 = 1.101 - (-1.099) = 2.200$$

The point on the x scale where log. x — log. 0.0796 = 1 is, therefore, 2.200 units below the isopleth intersecting the x scale at a value for $x = 12.62$; but, as it is necessary to plot values of x, and not values of $\frac{x}{0.0796}$, the point must be found which indicates $x = 1$. Accordingly, the value of log. 0.0796 is subtracted by adding 1.099 to the point (2.200 units below the value of $x = 12.62$) where log. x — log. 0.0796 = 1. Marking this latter point 1, the values are scaled from the items in Column (8).

In any case, the numerical coefficients of any of the variables may be treated in a similar way. The method of working with exponents Mr. Green.

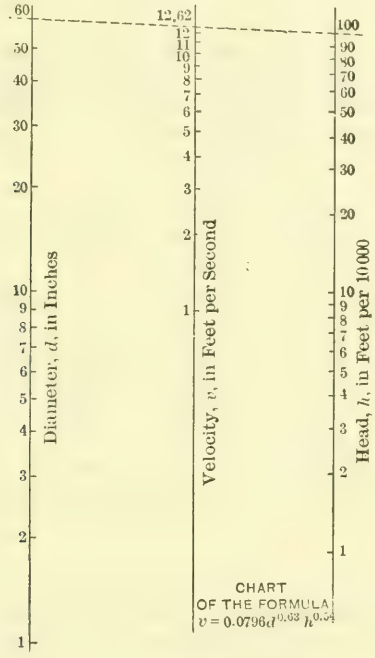

FIG. 47.

has already been developed quite fully. Fig. 47 is the chart made from the foregoing description.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

DISCUSSION ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE*

By Messrs. C. S. Reeve, John S. Miller, Jr., Clifford Richardson, J. E. Myers, Eugene W. Stern, T. Hugh Boorman, W. H. Kershaw, Philip P. Sharples, Robert A. Meeker, Hunter McDonald, T. M. Ripley, E. H. Thomes, R. A. MacGregor, E. M. Vail, W. B. Spencer, G. P. Hemstreet, Park A. Dallis, L. L. Tribus, Leonard C. Jordan, H. W. Durham, W. T. Headley, W. H. Connell, and C. R. Allen, Jr.

Mr. Reeve.

C. S. Reeve,† Esq. (by letter).—The writer has read the Committee's Report with interest and care, and finds himself in accord with many of its details. The solution of the various problems suggested on pages 3001 and 3002‡ would be of inestimable value to the worker in bituminous construction, and it is to be regretted that the conclusions reached by the Committee, thus far, could not be more specific in their helpfulness. As now stated, many of these conclusions have become almost axiomatic. It is believed that, as a whole, they constitute the best thought on the subject to date, but their statement in such general terms fails to yield the more exact information desired.

The writer hesitates to comment on the adoption of numerous "Definitions", as this is really a matter for the general policy of the Society. It cannot be seen, however, what credit for accomplishment can be ascribed to any of the leading technical societies for undertaking the publication of a general dictionary, or the definition of terms which in themselves are self-defined, as, for instance, "brick

* This is a discussion on the Progress Report of the Special Committee on Materials for Road Construction for 1914, presented to the Annual Meeting, January 20th, 1915.

† Chemist, U. S. Office of Public Roads, Washington, D. C.

‡ *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Reeve.

pavement", "side drainage", "drainage", "earth road", "sidewalk", "surface drainage", "stone-block pavement", "water-bound", "wood-block pavement", etc. It is believed that greater good can be accomplished, and greater attention commanded, by limiting such efforts to a glossary of disputed or unstandardized terms.

With reference to the specific definitions recommended, an inconsistency is noted in the use of the term, "200-mesh screen", under the definition of "Sand"; "200-mesh sieve" would better accord with the Committee's definition of "Sieve". In reference to this matter, the writer would favor having "fine aggregate" and "coarse aggregate" under "Aggregate", respectively, and analogous with "sand" and "gravel" under "Gravel", and then make the $\frac{1}{4}$-in. or $\frac{1}{8}$-in. mesh screen the dividing line between the two.

To be consistent with the definition of "Penetration", in which it is specified that the distance shall be "expressed in tenths of a millimeter", as later exemplified under the description of the test (page 3042*), the limitations set under "asphalt cement", "liquid, semi-solid, and solid bituminous material", and "hard and soft pitch", should be modified to accord with the method of expression recommended. For instance, 350 should be 35.0 mm., if the writer interprets correctly the intent of the Committee.

Considering Appendix C, the Committee is to be commended for its progressive attitude on methods of testing and its effort in general to assist in bringing about greater uniformity, by the recommendation, in many cases, of methods which, through recent years, have become quite well established. The seal of official approval on the bulk of this portion of the report would be a long step toward that uniformity which is so much needed and to be desired. In the way of a suggestion, the writer may state that the Office of Public Roads is now adopting the "Distillation Method" recommended by the Committee, with a slight modification in the form of condenser. The long slanting condenser is replaced by a short vertical water-cooled one, which helps to make the apparatus more compact, reduces the lag in the distillate at the time of cutting, and enables the operator to detect solids in the distillate much more quickly, by their rapid deposition on the water-cooled surface. Such solids can then be readily melted into their respective fractions by simply siphoning warm water through the condenser.

The attention of the Committee is called to an apparent error in the recommendation of "dimethylamine" for checking thermometers. "Diphenylamine" is probably intended, although the writer has recently adopted "benzophenone" for this purpose.

The writer believes that the method recommended for "Fixed Carbon" is open to criticism, unless possibly the Committee may have

* *Proceedings*, Am. Soc. C. E., for December, 1914.

supporting data to controvert any criticism. The preliminary heating, previous to the heating for a period of 7 min. under standard conditions, is probably recommended for the purpose of avoiding the foaming which takes place in testing certain types of product. Any efforts in the direction of obtaining an accurate determination on materials of this character are to be commended, but radical changes from commonly accepted standards should not be approved without a full knowledge of the effect of such changes on the results obtained. The same applies to the Committee's action in adopting new standards of quantity, container, and method of heating for the "Loss on Evaporation" test. Mr. Reeve.

The writer offers these comments and criticisms in a spirit of helpful interest, as it is only by hearty and sympathetic co-operation that we can hope to accomplish the ideals of uniformity so much to be desired.

JOHN S. MILLER, JR.,* ESQ. (by letter).—The Committee, on page 3001,† "urges resistance to a tendency, apparent in some instances, to ignore the opportunity offered by the Committee for collaboration, which would result in mutual benefit, and deprecates any tendency to return to the former chaotic conditions which would ensue from individual, non-concurrent effort". If two or more societies are working along the same lines, each wielding a powerful influence in its own sphere, and their recommendations as to nomenclature, specifications, and methods of test, are at variance with each other, then the "former chaotic conditions" referred to will be bound to continue, for the individual engineer will be governed to a great extent by the recommendations of that society with which he is most closely affiliated. Mr. Miller.

Of the eleven recommended methods of test in Appendix C, at least three are at variance with usual practice. Although more extended investigation may possibly show their value, because of the more accurate results obtained, confusion is bound to arise by the adoption of such methods before sufficient time has elapsed to determine definitely the relationship of the results obtained under the new method to those which are older and more generally used.

It is generally believed that to obtain concordant results on solubility in carbon tetrachloride, the solution must stand in the dark prior to filtration.

It would be very interesting to know why, in determining the "Loss on Evaporation", the sample must be stirred at all. Has the modification been introduced merely because it showed a higher loss, or were actual service tests made to compare the results with those obtained by this new method?

---

* Chemist, Barber Asphalt Paving Co., Maurer, N. J.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Miller.

There is also a radical departure in the method for determining fixed carbon. This test has always had its value when all necessary precautions were taken, and to modify it so radically destroys an accumulated mass of data which would still be available under the old method. An engineer adopting these radical changes cannot realize what they mean to those who, from the producing standpoint, have to interpret the specifications.

These criticisms are not made with any idea that present methods are ideal or that the present constants, both chemical and physical, tell all that we want to know. The writer firmly advocates any movement that will tend to make conditions of testing uniform and eliminate some of the present specification chaos, but doubts the value of suggesting such changes before their true value is known, both from the standpoint of what they mean as to quality or possible service tests, as well as means for identification.

Turning now to the Committee's general deductions as to construction and definitions, one finds several that are not clear or are contradictory.

The general deduction is made (page 3007)* that "The adhesiveness, cohesiveness, resiliency, and elasticity of the material should have the greatest possible permanence", and yet no methods are given for determining these valuable characteristics.

On page 3008,* under "Penetration Method", it is stated:

"An important factor for successful results is the proper compaction by rolling of the road metal before the spreading of the bituminous material."

What does proper compaction mean? If ultimate compaction is meant, then the definition for "Penetration Method", on page 3016,* is faulty. This reads:

"The method of constructing a bituminous-macadam pavement by pouring or grouting the bituminous material into the upper course of the road metal before the binding of the latter has been completed."

The question of amount of compaction before pouring the bituminous material is a veritable dependent on several physical characteristics of the bituminous material used. The writer certainly agrees that ultimate compaction finally is most desirable.

The qualified statement that sufficient data do not yet seem to be at hand to determine exactly in any case the value or possible advantage of using bituminous material in excess during construction, is well made. Certainly 3 in. penetration macadam has been constructed and has proved successful where quantities in excess of $2\frac{1}{2}$ gal. per sq. yd. have been used, but what would be a desirable maximum will be difficult to ascertain, as it depends not only on the type of bituminous material, but also on the size and character of the stone.

* *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Miller.

Under "Definitions" (page 3011)* we find:

"Asphalt Block Pavement.—One having a wearing course of previously prepared blocks of asphaltic concrete"

but no definition is given for asphaltic concrete.

"Asphalt Cement.—A fluxed or unfluxed asphaltic material, especially prepared as to quality and consistency, etc."

How can a straight-run or fluxed material be prepared as to quality, other than the inherent qualities of the materials used?

On page 3014,* we find "Cut-Back Products" defined, but there is no definition for "Straight-Run" except under "Pitch", and this, in turn, is defined as "solid residue produced in the evaporation or distillation of bitumens, the term being usually applied to residue obtained from tar."

Although "Dust" is defined, its definition, from the standpoint of a mineral filler, is not given except in a very general way under "Filler". Why not be as specific regarding this as for "Gravel", "Sand", "Silt", or "Clay"?

Although "Fat" is defined as "containing an excess", it has not been generally used in the asphalt industry. "Rich" is the term generally used for an asphalt mixture in which the asphalt cement is in excess and the excess is apparent.

"Free Carbon" is not always determined with carbon disulphide as the solvent; in fact, the writer believes that a mixture of benzol and chloroform is more generally used.

Why not define "Pea Gravel" more definitely, as in the preceding definition for "Gravel"? Surely "peas" commercially are variable enough in size to make this very ambiguous.

"Grit": Should not the word "chips" appear after the word "stone", and why could not this be more clearly defined as to size?

"Refined Tar" is defined, but there is no reference to "Filtered Tar". This should be covered.

The definition of "Rock Asphalt" is "sandstone or limestone naturally impregnated with asphalt". It is frequently impregnated with a soft material of the nature of maltha. The term, "Maltha", is not defined at all by the Committee, which seems to be an omission. *Circular 93* of the United States Office of Public Roads recognizes this term, and if its definition is not satisfactory, it should be corrected.

We find "Viscosity" defined, whereas the method for viscosity says that the results shall be expressed as specific viscosity compared with water at 25° Fahr. Why not define "specific viscosity"?

The cited instances of omissions and commissions are given merely to assist, if possible, in making the work of the Committee more satisfactory.

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* *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Miller.

If co-operation could be arranged between the committees of all the societies working on the same problem, it would have a tendency to eliminate many of the present "chaotic conditions".

Mr. Richardson.

CLIFFORD RICHARDSON,* M. AM. SOC. C. E. (by letter).—This report shows a marked improvement over any previous efforts of the Committee, although exceptions may be taken to certain statements to which the writer will refer.

The statement that progress has been steadily, if somewhat slowly, made in sheet-asphalt work is, of course, incorrect. This industry was well developed, and on a rational foundation, as long ago as 1896, when the well-known and highly successful sheet-asphalt surface was laid on Fifth Avenue in New York City. There has been some development of this type of pavement since that time, by the introduction of a close instead of an open binder, but the principle involved in the original pavement has not been changed, except in so far as it is now understood that the success of a surface of this type for pavements and roads is dependent on the surface area presented by the mineral aggregate for bitumen to adhere to, as shown by the writer,† and more recently interpreted in the light of modern conceptions of dispersoid chemistry. Unfortunately, for reasons of economy, and because the lowest bid of all tenders must generally be accepted, the principles which have been laid down are followed in too few instances in the United States; but in Great Britain there is a decided appreciation of the necessity of doing the highest type of work to obtain successful results.

It seems to the writer also that the Committee is going rather too far in stating that it is carrying out the first systematic and comprehensive work which has been undertaken in the endeavor to place the use of bituminous materials in highway work on a scientific basis. It seems to ignore the work of public officials, such as those of the United States Office of Public Roads, and many individuals. It is not an attitude that appeals to the writer as a fortunate one, especially in view of the fact that in an industry which has developed to such a limited extent at the present time, no fixed principles can be laid down with entire safety.

Many of the points discussed by the Committee are controversial, and it seems to have felt that it is justified in settling such controversial subjects without reference to authorities in other countries, and without sufficient study of the questions at issue. Some of the assertions consist of mere commonplaces, and, in other instances, of the use of terms which are not defined or their meaning interpreted, such as "The adhesiveness, cohesiveness, resiliency, and elasticity of the material should have the greatest possible permanence."

* New York City.

† "The Modern Asphalt Pavement," First Edition, 1905, p. 347.

Mr. Richardson.

This statement, no doubt, is true, but its interpretation is difficult, as the terms "adhesiveness", "cohesiveness", "resiliency", and "elasticity" are not defined in any way, and especially from the point of view of the physicist.

The statement is made in connection with the construction of bituminous surfaces that "mixing machines should be used, and hand-mixing methods should be avoided wherever practicable". The reasons for this are not given. Is it for financial reasons, or because better work may be done with machine mixing? In the writer's experience, roads built by the hand-mixing method have been as satisfactory as by any other; as, for example, Western Avenue, in Gloucester, Mass., which was constructed in 1909, and has cost nothing for maintenance up to the present time.

It is stated that "too heavy a roller should not be used". This is a mere assertion, and no explanation for it is offered. Undoubtedly, too heavy a roller can be used on a thin course of small stone, but it is questionable if this is the case with 2½-in. stone and a 3- or 4-in. course. Such statements as the preceding, without any explanation, are open to serious criticism.

Many similar assertions are made without offering any justification therefor. One is: "The use of fine sand on top of the bituminous material is open to grave objections * * * ." This, of course, is true, but it seems to the writer that the reason should be given in order that the assertion may be intelligible to the ordinary individual of little experience. It is, of course, owing to the instability of sand without filler when mixed with bituminous material.

It is stated:

"Trap rock in sizes greater than that passing a 2-in. screen should be used with caution in the construction of the upper course unless voids of the same are properly reduced, because of the likelihood of the individual stones to rock under traffic."

The writer is not in accord with this statement, because it has been found that, with due regard to the thickness of the course, the larger the stone used the more stable the course, and that, without a bituminous binder, the filling of the voids with graded material before compaction does not bring about stability, but the reverse, as it tends to force the larger stones apart and prevent their bearing on one another, which is necessary for stability.

It is asserted that: "An important factor for successful results is the proper compaction by rolling of the road metal before the spreading of the bituminous material."

This is altogether too general. It may undoubtedly be necessary with bituminous material which is excessively fluid when used, but, with the native asphalts, experience has shown that the best results are

Mr. Richardson.

obtained by applying them to the spread stone before it has been compressed at all.

The Committee believes that "the use of bituminous materials in quantities of more than 2½ gal. per sq. yd., where the upper course of macadam is to be 3 in. in thickness after compaction, is inadvisable under the penetration method".

Experience in Massachusetts has shown that the empirical rule of 1 gal. per inch of thickness of stone, excluding the seal coat, can be followed with satisfaction. If there is a slight effusion of bitumen in hot weather, this can be readily taken up with screenings, to the benefit of the surface itself.

The Committee presents a large number of definitions of road terms, but does not define all those used in its report, such as the terms "adhesive", "cohesive", etc., to which reference has been made.

It is unnecessary for the writer to repeat here his objections to including pyrogeneous hydro-carbons under the term "bitumen". There will be no general world agreement with the Committee's definition.

The Committee's definition of "Asphalt Cement" hardly seems to the writer to be satisfactory. It states that the material must be "especially prepared as to quality and consistency". The use of the word "quality" seems meaningless without further definition. Asphalt cement is further defined as being "suitable for direct use in the manufacture of asphaltic pavements" which would necessitate the inclusion of a bituminous broken-stone road under the classification of bitumens, which is questionable.

A "bituminous material" is defined as one "suitable for use as a binder, having cementing qualities, etc." Some statement should be included as to what it is a binder for. The definition evidently means a binder for road surfaces.

A "Bituminous Macadam Pavement" is limited by the definition of the Committee to one constructed by penetration methods. The writer can see no reason why a similar surface constructed by the mixing process is not a bituminous macadam pavement.

Definitions are proposed for "liquid", "semi-solid", and "solid" bituminous materials, but nothing is said about semi-liquid material.

The Committee fails to define asphaltic, semi-asphaltic, and paraffin petroleums, to which should probably be added a definition of semi-paraffin petroleum. In fact, the Committee does not define "petroleum" at all, which seems to be an omission, as it uses the word frequently in the definitions.

The Committee defines "Cut-Back Products" in relation to petroleum or tar residuum, but fails to give a definition for "Straight-Run Products", except as related to pitches.

The definition of "Cement" proposed by the Committee is an extraordinary one, but its interpretation depends somewhat on that of the

word "adhesive" which is used in the definition. Here, the general fault of definitions is observed, that is, an undefined term is used very loosely. An individual falling back on this definition would read "an adhesive substance", and would then seek a definition of the word "adhesive", which he would not find. In the writer's opinion, cement should be defined under two headings, namely, Cement—Bituminous and Cement—Hydraulic, and no attempt should be made to have a general definition of the two. Mr. Richardson.

The Committee's definition of "Clay" the writer hardly thinks would meet the approval of the geologists and soil authorities. Mr. George P. Merrill says* that

"It [clay] may perhaps be defined as an indefinite admixture consisting largely of more or less hydrated aluminous silicates and free silica, with lesser amounts of iron oxides, carbonates of lime, and various silicate minerals which in a more or less decomposed and fragmental condition have survived the destructive agencies to which they have been subjected. About the only feature characteristic of all clays, is that of plasticity, when wet, * * * and the term is used only with reference to degree of comminution and plasticity, regardless of mineral nature or chemical composition."

The well-known geologist, Geikie, defines clay, as follows:

"The decomposition of feldspars and allied minerals gives rise to the formation of hydrous aluminous silicates, which occurring usually in a state of fine subdivision, are capable of being held in suspension in water, and of being transported to great distances. Those substances, differing much in composition, are embraced under the general term, Clay, which may be defined as a white, grey, brown, red or bluish substance, which when dry is soft and friable, adheres to the tongue, and shaken in water becomes mud. It is evident that a wide range is possible for varieties of this substance."

In the light of recent conceptions of colloidal chemistry, the writer would define clay as follows:

"Clay is a material resulting from the weathering of rock, usually feldspathic, possessing the property of plasticity when wet, which is due to the presence of disperse colloids. Clay may be residual, *in situ*, where it is formed, or sedimentary and transported by water. In its purer form, it consists of a hydrated silicate of alumina, but often contains quartz or other residuals of the rock in which it originates."

The Committee's definition of "Coal-Tar" seems to be quite inadequate, as no reference is made to the various derivatives of the hydro-carbons, which are characteristic of this material.

The writer is not in sympathy with the Committee's definition of "Consistency", at least, from the point of view of the physicist. He thinks that such a definition should refer to "Consistency" as being an indication of the degree of internal friction of the molecules of any material.

* "Rocks, Rock Weathering and Soils," p. 117.

Mr. Richardson.

In the definition of "Dust", the two classes should be differentiated by figures, as in that of the word "Flushing". The second class is defined as "the product of the crusher passing through a fine sieve". Some statement should be made as to the size of this sieve, if a sieve is to be accepted as differentiating such a product. As the finest sieve, 200-mesh, is that which is no doubt intended to be used, it should be recognized that such a sieve passes sand as well as dust, and that the latter can only be determined by elutriation with water.

"Earth Road" is defined, but no reference to the term, "Top-Soil Road", is made.

It is doubtful whether, in the light of present conceptions of colloidal chemistry, the definition of "Emulsion" is a sufficient one.

The Committee attempts the definition of the term "Fat". It is unfortunate that it should sanction the use of such a word, especially in view of the fact that this use is not a common one. In the writer's experience of more than 27 years in the industry, he has no recollection of its common use. The term, "Rich", is usually chosen in its place.

"Free Carbon" is defined as "organic matter which is insoluble in carbon disulphide". Other solvents are frequently used in practice, such as chloroform, benzol, and a mixture of benzol and toluol.

The writer can hardly agree with the definition of "Gravel" proposed by the Committee. Very general practice has placed the limit of the size of the smallest material in gravel at that retained on a sieve of 4 meshes to the inch, and the writer can hardly believe that making it a 10-mesh sieve will be acceptable.

"Grit" is defined as "stone, slag chips, etc." The word "chips" should be added after "stone".

The Committee gives no definition covering the type of sands found in Kentucky, which are impregnated with maltha. Such a material would not be included under rock asphalt.

The definition of "Loam" would be improved by following that of Geikie, which reads: "An earthy mixture of clay and sand with more or less organic matter."

As a substitute for the Committee's definition of "Macadam", the writer would propose the following:

"A term descriptive of a broken-stone road, derived from the very euphonious name of a well-known highway engineer and pioneer in road construction, which can readily be used as a noun, adjective, or verb. As a noun, it is applied to the broken stone of which a road is built, or to the road's surface itself, 'paved with macadam'; as an adjective and as a verb, its uses are evident.

"A more suitable designation for macadam is 'a broken-stone road'."

"Mastic": In the light of the writer's experience with this material, it is not defined satisfactorily by the Committee. The character-

Mr. Richardson.

istic of this material is that it contains more than sufficient bitumen to fill the voids in the mineral aggregate, which enables it to be floated into place, in distinction from other bituminous mixtures, with which rakes are used. The following definition is proposed:

"A mineral aggregate consisting of a large proportion of filler, sand, and grit or gravel, to which an excess of asphalt has been added, more than sufficient in quantity to fill the voids, and which can be put in place with a float."

The writer objects to the term "solidity" in the definition of "Penetration". "Solidity" is not defined by the Committee, and its use is meaningless. It is suggested that the sentence should read: "The degree of internal friction of bituminous material".

The definition of "Sand" will apply well to that material as used in sheet-asphalt pavements, but, in hydraulic concrete, it would include particles passing a screen of 4 meshes to the inch, according to the usual practice.

To the writer, the definition of "Sand-Clay Road" seems to be incomplete. The intimate mixture of sand and clay must be regulated (and it is believed that this should be stated) so that the clay fills the voids in the sand, or only to a very slight excess.

The definition of "Screenings" is very loose, in stating that they will pass through a $\frac{1}{2}$ to $\frac{3}{4}$-in. screen. A statement should be given of the size of opening through which the particles would pass, as the size of those passing a screen will depend on the inclination of the screen and the rate at which it is fed.

In the definition of "Silt", it should be stated from what a "naturally fine earthy material" is deposited, and how. There are many sands which will pass a 200-mesh sieve, which are not silt. It would seem that there must be some further limitation in this regard. From the use of the verb "to silt", as "a stream silts up", it would seem that this would apply generally to deposition from water.

The Committee's definition of "Viscosity" is unsatisfactory, from the point of view of the physicist, in the same manner as its definition of "Consistency". Any textbook of physics would afford a more satisfactory definition than that which is given.

Although the definitions of the Committee do not show the influence of an experienced lexicographer, and are open to further criticism than that which the writer has given them, and though, for an important Society like this, they are not anything to be proud of, it is possible that their recognition may aid, in so far as they go, to interpret the language of the Committee.

Any report of a committee is more or less tentative, and can never be final in regard to subjects which are progressive, but it serves an immediate purpose as a guide and for educational purposes, and for this reason, this report will, no doubt, be of value.

Mr. Myers.

J. E. MYERS,* ESQ.—In the methods of "Analyses and Tests of Bituminous Materials", on page 3040,† under the heading "Consistency" it will be noticed that 4° cent. is one of the temperatures at which it is recommended that the consistency be taken. Under the heading "Float Test", it is stated, on page 3042,† that the collar and plate be placed in a bath at 5° cent. As it is very difficult to maintain a bath at this temperature within a range of one degree, it would seem advisable to change one of these temperatures so that both will be the same. Also, under the same heading, it is stated:

"The time, in seconds, between placing the apparatus on the water and when the float sinks, shall be taken as a measure of the consistency of the material under examination."

In the printed instructions received with the instrument for making the float test, it is stated that as the water gains entrance to the saucer the entire apparatus sinks below the surface of the same, thus explaining the use of the word "sinks". In *Bulletin No. 38* of the U. S. Department of Agriculture, Office of Public Roads, in which is given the "Methods for the Examination of Bituminous Road Materials", under the heading "Float Test" is this statement:

"The time in seconds between placing the apparatus on the water and when the water breaks through the bitumen is determined by means of a stop-watch and is taken as a measure of the consistency of the material under examination."

Thus it will be seen that the points between which the time is recorded, are defined in at least two different manners. There is a difference of from 3 to 5 sec. between these two methods of conducting the test. In the Committee Report it would seem well to add "below the surface of the water" after the word "sinks" so that the paragraph would read:

"The time, in seconds, between placing the apparatus on the water and when the float sinks below the surface of the water, shall be taken as a measure of the consistency of the material under examination."

In the paragraph under the heading "Paraffin", page 3049,† the first sentence reads: "One hundred grammes of the material shall be distilled rapidly in a retort to a dry coke". This description of the apparatus and method used for making the distillation is not definite enough to insure the use of like methods and apparatus in various laboratories. Also, the expression "distilled rapidly" is too broad a term. It would be better to place a fixed maximum limit on the time of distillation. It has been the experience of the laboratory of the New York State Highway Department that the determination of paraffin is one of the most unsatisfactory and difficult operations

* Chf. Chemist, New York State Highway Comm., Albany, N. Y.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

in connection with bitumens. Part of the difficulty may be disposed of by an accurate description of the apparatus used in making the distillation. This should state the kind of retort used, as iron, glass, or copper; its shape, dimensions, position of outlet, and position of the retort in relation to the flame during the distillation. In any given style of retort, the quantity of paraffin which can be recovered will depend largely on the rapidity of the distillation. This factor becomes more or less fixed by placing a maximum limit on the allowable time for the distillation. Mr. Myers.

In making the distillation of a bitumen to a dry coke, there is a loss of material due to "cracking". As a result of this, the percentage of actual distillate obtained plus the percentage of coke will not equal 100 per cent. In calculating the percentage of paraffin to the original material, different results are obtained, depending on whether this loss of material is considered or neglected. It would seem advisable, therefore, to give in more detail the method to be used in calculating the paraffin. In the description of the freezing method and apparatus used, it would be well to add that the funnel and filter paper through which the alcohol and ether solution are to be filtered, should be surrounded by a freezing mixture of salt and ice.

Some of the points mentioned are of minor importance, but it is only by the strict observance of these little details in the methods of performing tests that the personal equation is largely eliminated and the work of different operators and different laboratories is made to harmonize.

Eugene W. Stern,* M. Am. Soc. C. E.—The speaker is disappointed in finding that this report is confined mostly to bituminous materials. The Committee has devoted much time and thought to the subject, and the report is unquestionably of value; but the Society should soon have an authoritative report on the various types of road and street pavements now in use. Mr. Stern.

The speaker should not be considered as criticizing the Committee. It has done its work faithfully, but he thinks that it should have a larger membership, so that it could be divided into sections, one to report on stone block, one on brick, one on wood block, one on concrete, one on bituminous pavements, etc.

It is estimated that $1 000 000 per day is being spent on roads and streets in the United States, a very large proportion on road surfaces other than bituminous, and still nothing has been done by the Society to place in the hands of its members something authoritative on such materials.

It is important that prompt action be taken by the Society, and it is hoped that an enlarged committee will be appointed to submit

* New York City.

Mr. Stern. a preliminary report not later than September, 1915, in order to enable members to discuss it thoroughly before the Annual Meeting, in January, 1916.

Mr. Boorman. T. HUGH BOORMAN,* ESQ.—Mr. Miller objects to the definition of "Rock Asphalt" given in the report. Rock asphalt, from the earliest days of asphalt pavements, has been generally known as the European limestone. American sandstone is comparatively new, and, of course, has been connected with American asphalt almost entirely. Probably Hungary is the only country in Europe where sandstone asphalt is found.

The term "Maltha" has been referred to. This is a proper term, and is applied to "liquid asphalt".

Mr. Richardson has criticized light rolling, but all will agree that, on asphalt surfaces, a light roller is required—not a heavy one.

In several places in the report, the use of slag as a mineral aggregate is mentioned. Attention is called to the following statement† by James Owen, M. Am. Soc. C. E., before the Good Roads-Automobile Convention, at Atlantic City, N. J., on September 25th-26th, 1908:

"First, we must abandon the idea that it is necessary to have hard roads; secondly, they must be made elastic by the interjection of a suitable material; and thirdly, they must wear but little. The experiments with tar show that a soft stone is better than a hard one. Slag, limestone, granite, or trap will be the order of preference."

At this Convention, in commenting on the statement of the Deputy State Highway Commissioner of Pennsylvania, on the scarcity of broken stone in his State, the speaker stated that he had arranged for laying an experimental pavement with asphalt and slag. Since that time little has been done in America in regard to the utilization of slag in road building.

In 1911, in the course of research work in England, the speaker found that "Tarmac" was the most extensively used material for road construction in that country. All American highway engineers and experts who attended the last International Road Congress in London have expressed their satisfaction with this system of road construction. Owing to the tremendously increased production of slag in the United States, new outlets for its use have become desirable, and it is believed that steps are now being taken to erect large works with a contemplated output of possibly 1 000 tons per day, to be treated under the Hooley patents with a bituminous penetrator and binder.

The strength of slag as compared with trap rock is shown in Table 1, which gives the results of tests made recently at Columbia University.

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* Cons. Highway Engr., New York City.

† "What a New Jersey Engineer Would Do."

TABLE 1.—Comparative Tests of Crushing Strength of Trap Rock and Slag, Made at Columbia University. Mr. Boorman.

Trap Rock.				Slag.			
Weight of sample.	Area, in square inches.	Load at which failure occurred, in pounds.	Equivalent to pounds per square inch.	Weight of sample.	Area, in square inches.	Load at which failure occurred, in pounds.	Equivalent to pounds per square inch.
72 lb. 4 oz..	50.26	88 932	1 770	65 lb. 12 oz.	50.64	119 860	2 370
73 lb.....	50.90	107 870	2 120	66 lb. 8 oz.	51.23	116 650	2 275
73 lb.....	50.52	103 180	2 045	66 lb. 6 oz.	50.52	123 870	2 470
71 lb. 14 oz.	51.02	101 400	1 991	67 lb.....	51.02	140 310	2 750
Average.....			1 987	Average.....			2 466

Trap concrete weighed 154.5 lb. per cu. ft.
Slag " " 140.6 " " " "
Trap weighed 100 lb. per cu. ft.
Slag " 80 " " " "

On bituminous pavements, at the shoulders and along car tracks, it is quite a problem to provide for places where the pavement comes in contact with softer materials. In Washington, D. C., where a bituminous pavement is laid on each side of the car tracks, very injurious effects have been noticed at the junction. There, unfortunately, as in so many cities, the railroad company is responsible for the pavement between the tracks and for 2 ft. outside, and this outer 2-ft. strip soon becomes badly disintegrated where it adjoins the other pavement (or at the shoulders). The speaker is of the opinion that the Committee might take up a study of this matter and suggest a remedy.

In reference to the formation of waves or corrugations in bituminous pavements, this can generally be traced to an excessive quantity of light oil in the mixer, caused mostly by too much flux. The speaker's attention has recently been called to an asphalt pavement on Broadway, New York City, in which the individual blocks have been actually curved. How was that caused?

In reference to the thickness of concrete foundations for city streets, particularly under the new conditions of very heavy auto-car traffic, it would seem well to adopt the general method of European construction, which requires a thickness of from 8 to 9 in. Attention is called to the pavement on Fifth Avenue, New York City, in which the concrete averages 12 in. in thickness. The City of London proper, requires a 9-in. concrete base, and the Borough of Westminster, which, in 1912 allowed a 6-in. concrete base, in 1913 insisted on a 9-in. concrete foundation. In Berlin, the concrete base is required to show,

Mr. Boorman. at the end of 19 years of maintenance, a thickness of at least 7.9 in. In Paris, where there is only ordinary light traffic, the thickness must be about 6 in., but, in most cases, where any heavy traffic is encountered, the engineers call for about 8 in.

Mr. Kershaw. W. H. KERSHAW,* ASSOC. M. AM. SOC. C. E.—To the investigations mentioned on pages 3001 and 3002† to be undertaken by the Committee, the speaker would add the formulation of a standard method of taking samples for analysis. This is an important matter, and should be handled so that in the Committee's next report it would be in such shape that it could be generally adopted. It should be borne in mind that the man who takes the sample does not know the chemist's requirements; that in the case of cut-backs he might heat the sample before he takes it; or that it might or might not be taken from the center of the barrel or tank car. All the details should be specified, and the instructions should be explicit as to the quantity required, the size of the container, etc., for different materials.

These ideas are mentioned simply so that others may offer suggestions for the guidance of the Committee, and so that complete instructions as to the methods of obtaining samples may be adopted.

Mr. Sharples. PHILIP P. SHARPLES,‡ ESQ.—The Committee's report is admirable as a report of progress; a comparison of its reports for several years is very instructive, as it shows the advance made in paving work of all kinds. The details must be studied, so that they may be amplified and corrected. The criticisms of the report have been excellent.

In building a penetration road, the rolling and preparation of the surface for the bitumen is of the utmost importance, and that subject should be considerably amplified by the Committee. The engineer should strive to obtain a surface open enough for the bituminous material to enter readily, and, at the same time, with the interstices reduced to such an extent that the bituminous material, when it enters, will be held in the required place. It is difficult to obtain good results with all kinds of stone. A specification which details exact methods for a hard trap rock, would cause failure with soft limestone. However, if the principles which govern penetration work could be explained, better results would be accomplished than by an arbitrary specification.

Mr. Fulweiler's remarks as to the condition of the surface necessary for the success of surface treatments are endorsed. Some of the figures given at Atlanta, Ga., by Col. Sohier, on the work of the Massachusetts Highway Commission on surface treatments, are very valuable in the interpretation of results obtained with such treatments.

* New York City.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

‡ Mgr., General Tarvia Dept., Barrett Mfg. Co., New York City.

Mr. Sharples.

Many subjects mentioned in the report require further research, for example, those on pages 3001-3002,* and at the top of page 3010,* should furnish much food for thought. Those who set out to solve these problems would do well to read the report with great care and use it as a basis for further experimental work.

The tests detailed in Appendix C are of great importance. The speaker agrees with Mr. Fulweiler that the loss on evaporation has no place in the testing of tar materials. It does not afford any information which is not given better by the distillation method. However, if this test is to be carried on, the temperature should be much reduced. The common one used at present is 105° cent.

The speaker can corroborate Mr. Fulweiler's statement in regard to the determination of the consistency, by the penetrometer, of coal-tar materials containing any quantity of free carbon. It is difficult to obtain concordant figures, and some method based on the melting point furnishes a much more desirable test.

With reference to distillation, the Committee is correct in following the work of the American Society for Testing Materials, as the report of the Committee of that Society is very complete, and is as good a work on that subject as has ever been done. Mr. Reeve has stated that the Office of Public Roads is at present working on a method differing from that proposed by the American Society for Testing Materials. The speaker thinks that any change should be considered carefully, and not made unless some great advantage is to be gained. The report of the Committee of that Society on distillation has been widely accepted throughout the United States, and the method of distilling tar materials proposed is being used in almost all the large laboratories. Any change, unless of great importance, is deprecated. If any change is to be made, it would be well to have the matter discussed before the American Society for Testing Materials, in order to get the wide experience represented in that society.

In the distillation test, the melting point of the residue is called for. The melting point is important, but the specific gravity of the distillate is also important, and should be required in the list of tests, as it gives additional information regarding the source of the tar and the way it is manufactured.

In reference to consistency, the temperatures which have been adopted generally, for different classes of coal-tar materials, are 25°, 50°, and 100° cent. It would be a mistake to substitute new temperatures, as the others are so well recognized in specifications, and so many chemists have data thereon.

The speaker endorses Mr. Myer's remarks in regard to the paraffin test. It is very desirable to state definitely the type of apparatus to

* *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Sharples.

be used in distillation, if any important results are to be obtained.

Mr. Kershaw has mentioned another important point in reference to samples and the method of obtaining them, and this should receive consideration. Samples have come to the speaker's laboratory which have been improperly taken, and from which no conclusions as to the condition of the original material could be drawn. Samples have even been taken from heating kettles after the material had been drawn from tank cars, and after it had been heated for some time; and these have been submitted as true samples of the material. The importance of having the sampling done properly cannot be emphasized too strongly.

In reference to the causes of the formation of waves or corrugations in bituminous pavements, it is necessary to differentiate sharply between construction faults and waves due to traffic. Such waves may be seen on pavements laid on concrete foundations, with absolutely no possibility that they were due to faults in the foundation. Therefore, there must be cases where they are due to other causes. On the other hand, there are waves and corrugations which are undoubtedly due to the foundations, and the only part of the road affected is where the foundations are weak. In tracing the causes of these defects, the inquiry should be given a very wide scope and should take into consideration a great many different factors. There have been cases where pavements have been free from waves for many years. Some quite famous ones have been reported in Chicago, Ill. Then, under some sudden change in traffic conditions, waves have been produced.

At present, it is not possible, in every case, to state the cause, but it is known that heavy traffic, that is, heavy weights per unit of width of wheels, has a great influence on the formation of waves in a properly constructed pavement. When the traffic increases to a certain point, or when the weight goes beyond some definite limit as to width of tires, etc., these waves begin to form. It would be interesting to investigate their causes in bituminous pavements of various types. In a properly designed pavement, it is very largely a question of traffic for any given bitumen or any given class of construction. A change in bitumen will make a change in the character of the pavement and in its ability to withstand certain weights of traffic.

The subject of spraying machines is an alluring one to the speaker, as he has worked on it for many years. In designing apparatus for this purpose, many conditions must be considered, for an apparatus which is successful with one kind of material may not, necessarily, be successful with another kind.

The small quantities needed for surface treatment, up to ½ gal. per sq. yd., furnish the easiest problem. With binders, it is necessary to provide for large quantities, at least 1½ gal. per sq. yd., and with them comes the problem of lapping over. A lap made when putting

on 1½ gal. gives 3 gal. per sq. yd.; if the lap is missed, the road gets no binder at that place. Mr. Sharples.

The pressure is also an important feature, and, in any good apparatus, the amount necessary depends on the material and the way it is handled. A material which can be thinned by the application of a low heat will require less pressure than one which liquefies only at a high heat and must necessarily be put on at a somewhat lower heat than is necessary to complete the liquefying.

In trying to decide on the pressure necessary for different applications, it must be remembered that there are many kinds of nozzles, and the pressure necessary to force a material through one kind may be wholly inadequate with another.

Spraying apparatus is not yet developed to such an extent that it can be said that any one type will be efficient for all classes of work, although there are on the market heavy duty types which work very well for a great variety of materials.

ROBERT A. MEEKER,* M. AM. SOC. C. E.—On page 3005† of the report, the following statement occurs: Mr. Meeker.

"The crown generally used in the construction of broken stone roadways is excessive when bituminous materials are used, and a crown of even ½ in. per ft. should be avoided when a lesser crown can be secured without detriment to the surface drainage."

The speaker is glad to note that this statement is qualified. His experience would lead him to re-write this paragraph as follows:

"The crown generally used in the construction of broken stone roadways is excessive when bituminous materials are used, and should never be more than ½ in. per ft."

When an attempt is made to build a road with a crown, the form of which is the arc of a circle, and the cross-section has a proportionate slope of less than ½ in. per ft., by the time the road is finished, it will be found that, in some places, the slope is not ¼ in. per ft., and water will stand on the surface, even though the road has a concrete base. Concrete men know very well that concrete settles and spreads as it reaches its final set. The speaker had trouble of this kind on one road for which the crown was designed to be ½ in. per ft., but as the road was quite wide this was reduced to ⅜ in. per ft. As a consequence, before that road was 3 months old it had slight depressions in which water collected. Even on a sidewalk, for which the old rule was ½ in. per ft., there are occasional depressions in which water collects. Sidewalks having a slope of ¼ in. per ft. are often covered with water in large spots. For the protection of the road

* Plainfield, N. J.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Meeker.

and the general benefit of the users, that clause of the specification should be corrected.

At the bottom of page 3005,* the following statement is found:

"Where bituminous pavements are laid, the edges should be protected and a sudden transition from the pavement to any softer shoulder material avoided by means of cement concrete or other edgings and such reinforcement of the shoulder material as may be necessary."

The speaker thinks that is wrong. If a concrete shoulder is built along a bituminous concrete road it will cost about $4 000 per mile, and if that additional sum is made available, the width of the pavement can be increased sufficiently to provide room for everybody; it will provide a road much more useful and valuable to every person along the line.

Unfortunately, bituminous concrete crusts are sometimes laid on rather poor sub-grades, and are held in place with concrete curbs. A teamster driving down hill on such a road runs one wheel over on the side, just beyond the concrete shoulder. Then, if his load is heavy, he crowds the wheel against the curb, and the result is very injurious to the road.

On page 3007* there is a sentence which deserves special emphasis:

"Excessive sizes, or excessive variations in the size of the mineral particles, should be avoided, and the utmost care must be taken to avoid the segregation of particles of different sizes."

This sentence is the explanation of the greater portion of the so-called waves and irregularities in bituminous pavements.

On the same page there is the statement: "Too heavy a roller should not be used". This has been criticized by Mr. Richardson, and rightly. In using a bituminous concrete, containing stones that will pass a 2-in. ring, a heavy roller must be used or the pavement will fail, and fail in some places almost unexpectedly.

The speaker had occasion to examine a bituminous concrete road which had been built with 2-in. stone, and found in many places that the stones had bridged. Instead of being a packed mass with a minimum percentage of voids, it had in some sections as high as 40% of voids. That road had been rolled with a 5-ton tandem roller, longitudinally and also diagonally. A light roller produces a good-looking road, but in reality there is simply a thin crust on top and beneath it a mass with a varying percentage of voids. Such a road will not wear uniformly, but will have pot-holes and depressions, otherwise designated as waves.

The speaker is thoroughly in accord with Mr. Richardson's remarks concerning the use of a heavy roller on roads of bituminous concrete containing dry mineral aggregates. The first few trips of the roller

* *Proceedings*, Am. Soc. C. E., for December, 1914.

causes ruts and ridges, but, if the rolling is continued, these will disappear, the voids will be reduced to a minimum, the pavement will be uniform, and the road will last. Mr. Meeker.

On page 3007,* it is stated:

"Trap rock in sizes greater than that passing a 2-in. screen should be used with caution in the construction of the upper course unless voids of the same are properly reduced, because of the likelihood of the individual stones to rock under traffic."

The speaker is of the opinion that trap-rock greater in size than that passing a 2-in. screen should not be used, because, if great care is not taken, the stones will bridge and thus form voids which will cause depressions later.

On page 3009,* it is stated that: "The roadway surface should be dry when the bituminous material is applied". This has been repeated so often that it has been accepted as almost axiomatic. The speaker would rather have a road slightly moist and perfectly clean on which to apply a bituminous top, that one which is bone-dry and coated with thin fine dust. Such dust, in spite of anything that can be done, will separate the stones from the bituminous material, will make a loose top, and separate it from the base.

The speaker's attention was first directed to this matter on a penetration job, on which the contractor had been duly impressed with the necessity of cleaning the road before applying the bitumen. He had applied some, but the result was not satisfactory, the bitumen did not stick, and the work had to be done again. There was a little dust on the surface of the stone, and this he removed by sprinkling. Then, before the surface was thoroughly dry, he applied the bitumen. It was a good piece of work. Some of the pavement was taken up for inspection, and it was almost impossible to separate the bitumen from the stone.

Of course, it is known that the stone must be clean, but the speaker does not think it advisable to specify that it should be dry. The specification should state simply that the roadway surface shall be clean, and let that condition be obtained in the best and cheapest way.

In the definition of "Gravel" on page 3015,* there is a little discrepancy. Gravel is a very wide term, and has various meanings in various localities. Gravel used in road construction alone, without the addition of any other material, is very different from that used in making concrete or in preparing bituminous concrete. The definition given is:

"Gravel.—Small stones or pebbles usually found in natural deposits more or less intermixed with sand, clay, etc., but in which mixture, the particles which will not pass a 10-mesh sieve predominate, gravel

* *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Meeker. clay, gravel sand, clayey gravel, and sandy gravel indicate the varying proportions of the finer-sized particles. * * *"

Then follows the definition: "Pea Gravel.—Clean gravel the particles of which equal peas in size". The speaker believes that this definition would cause trouble if carefully followed.

On page 3005* there is a statement about "the proper treatment of a broken stone, gravel, shell, or slag roadway with bituminous material". If a clay gravel is treated with bituminous material, it will cause trouble. The speaker would advise that this be amended by inserting the word "clean" before the word "gravel", in the quoted sentence.

The speaker's experience indicates that the use of asphaltic oils for surface treatment on clay gravel roadways is deleterious rather than beneficial.

Mr. McDonald. HUNTER MCDONALD,† PAST-PRESIDENT, AM. SOC. C. E.—The speaker has had very little experience in the construction of roads, his work having been confined almost entirely to railroads, but, as there has been some criticism of the Committee's definitions, it might be well to call attention to the work of the American Railway Engineering Association. That Association has a rule that, when a material is to be defined by a committee, the definition must apply only to the use to which that material is to be put by those who are carrying out the work on which the committee is reporting. For instance, the definition of sand by the Masonry Committee is not sand at all, but "fine aggregate". It is not called sand. The definition of sand by the Ballast Committee is "any hard, granular, comminuted rock which will pass through a No. 10 screen and be retained upon a No. 50 screen". The rule is that the definition must be made from the standpoint of its use, and without regard to any definitions which may appear in dictionaries. If one man uses sand for making concrete and another for making ballast, their definitions are entirely different.

Attention is called to the omission of the definition of "chert", a road material which is quite common in the South and West. Perhaps the Committee, in a later report dealing with non-bituminous materials, will include a definition of this word. It is defined in the "Manual" of the American Railway Engineering Association as "an impure flint or hornstone occurring in natural deposits". In many of the States in the South and West, it is the best available material for roadway work, and makes a good road if properly treated.

Mr. Ripley. T. M. RIPLEY,‡ M. AM. SOC. C. E.—No hard-and-fast rule can be applied to the question of the proper weight of the roller to be used on bituminous roads, as it depends on so many variable conditions.

* *Proceedings*, Am. Soc. C. E., for December, 1914.

† Nashville, Tenn.

‡ Watertown, N. Y.

Mr. Ripley.

One extreme is where a road is built by hand-pouring; the other is where a machine is used with high pressure. Another factor is the hardness or softness of the stone.

The pressure machine which has been used on the speaker's Division gives a penetration so much greater than can be attained by any method of hand penetration that there is no comparison, and, on such roads, heavy rollers may be used.

The speaker is using the stone shoulder on several roads in his Division, but has not used concrete edging except for brick pavements, and has not even considered it because of its cost. A teamster can run out on the stone shoulder as much as necessary for vehicles to pass one another, and such a shoulder can be repaired and taken care of far better and far easier than a concrete edging.

With reference to the formation of waves in pavements, the speaker believes this to be due principally to faulty foundations. With a good foundation, properly rolled, and in smooth condition, on which to place the asphaltic top, many of the so-called waves would be eliminated. Another cause of waves is improper pouring, due to the consistency of the oil. On several thousand miles of roads traveled over by the speaker during the past summer, on his own and on other divisions in New York State, he believes that most of the trouble has been caused by badly prepared foundations or by applying the bitumen improperly.

On the speaker's work, in the majority of cases, the pressure has been usually about 30 lb., but is increased to 40 lb. when the material is colder than called for. It has been found that a large proportion of the bituminous material goes into the bottom of the top course. On the greater part of the State work, the top course is 3 in. thick. On digging up some of that work, it was found that the bottom of that course was thoroughly filled. Possibly this may be detrimental to the wearing surface, and that more should be poured on the last or squeegee coat.

The asphalt, also, in many instances, has been poured at a much lower temperature than could be done by hand, and, at the same time, it has been forced down among the stone. That is a very important matter, at times, particularly in the northern part of the State, and possibly in all parts of the State, except in the Long Island District, where the fluctuations of temperature are not so great as in the Central and Northern Districts.

On a short piece of road in the Adirondacks, late in November or early in December, the speaker found it necessary to pour a small quantity of bituminous material, and the use of the pressure machine was of great advantage.

Ultimately, the speaker believes, it will be necessary to write a clause in the specifications relating to the minimum pressure to be

Mr. Ripley. used, but, of course, that pressure will depend on the nature of the bituminous material, and probably on three or four other factors.

The machine used on the speaker's Division was built by a machine company, and was improved in two or three respects by the contractor using the first one, which made it a practical success.

The speaker is undecided as to whether the best road is produced with the greater part of the bituminous material in the bottom course, or whether it would be better to have more on top. This can only be decided by observations covering some time.

The problem of overlapping has been mentioned, but if the man doing the pouring is expert, there should be no trouble. The delivery from the line of nozzles is almost absolute, and a good man can start within a fraction of an inch of the proper line. The speaker was surprised to observe how close to the old pour the new one was started by a man who had become expert in using the machine. It has certainly been a great help in getting a uniform distribution of the bituminous material in penetration roads.

Mr. Thomes. E. H. Thomes,* M. Am. Soc. C. E.—It would seem that a ¼-in. screen or sieve should be the dividing line between sand and gravel, instead of a 10-mesh sieve, as stated in the definition of "Gravel". Although a 10-mesh sieve is the maximum ordinarily used in sheet-asphalt paving, a ¼-in. size is more commonly used as the maximum for sand in concrete and in cushion beds for block pavements. The grading of the sand for asphalt pavements, grouting, etc., can be specified by the size of the sieves.

It is assumed that the screens mentioned in the Committee's report are laboratory screens. In the report or in the specifications, attention should be called to the fact that, in making laboratory tests of mineral aggregate, and also other materials, there should be slight allowances for small variations due to commercial conditions, the unreliability of small samples and methods of taking them, and the personal equation in making the tests.

Where a material passing a ¼-in. screen is specified, cases arise in which a sample tested in the laboratory is found to contain some larger particles. Sometimes a report is made simply that the material does not comply with the specifications, and the material is sometimes unfairly condemned on one person's test of a small sample. To obtain a ¼-in. material, it is necessary for a screening plant to use larger screens, and sometimes openings of different shape in order to allow for variations due to dampness and to local and commercial conditions.

In the matter of the crown of a road, mentioned by Mr. Meeker, it would be well to consider the effect of automobiles on roads with a high crown. Such a crown tends to incline a larger part of the weight

* Jamaica, N. Y.

Mr. Thomes.

on the wheels toward the sides, and, in bituminous pavements, produces waves, especially when the edges are not supported. An unsupported edge may remain in good condition for several years and then become wavy, comparatively rapidly. The trouble with hard stone, brick, or concrete shoulders is that they are likely to cause a rut along the inner edge, which increases the cost of maintenance. A shoulder which is level with the pavement, and will wear uniformily with it, is better. A concrete shoulder or edging is good, but in many cases it is more advisable to form a shoulder of broken stone along the edge of a bituminous pavement, and roll both together. The voids in the broken stone can be bonded with screenings, or better with bituminous binder applied by a hand pouring pot. A broken stone shoulder, several feet wide, can be made at the same cost as a brick or concrete shoulder, and give that much additional roadway. The width of the roadway and the character of the shoulders should be governed by the traffic and other local conditions. The crown will vary slightly, according to the kind of pavement and the drainage conditions.

The best and simplest rule for the transverse section of a pavement is to drop one-eighth of the crown (or the difference in elevation between the center and the gutter) at a point one-quarter of the distance from the center to the gutter; drop one-third of the crown at one-half the distance, and drop five-eighths of the crown at the three-quarter point. The circular and parabolic sections for ordinary widths give 0.06, 0.25, 0.56, and 1.00 of the crown drop at the four quarter points from the center to the gutter. The former rule gives 0.12, 0.33, 0.62, and 1.00, or a little more slope from the center, where it is needed.

For the average narrow country road, it is only necessary to drop one-third of the crown at one-half the distance between center and side. It is impracticable to use an office table calculated to thousandths of a foot. A template should be used wherever possible.

The manner of rolling, and the character of the rollers, have considerable effect on the wearing surface obtained. With asphaltic pavements, better results seem to be obtained by using a light roller first, and then a heavier one, and by rolling transversely and diagonally.

The speaker thinks it would be well for the Committee to include in its report remedies for overcoming waviness.

The creeping of asphalt blocks is prevented by the use of a course of reinforced anchor-blocks every 15 ft., more or less. In these blocks is embedded a thin steel plate of elliptical shape which projects $\frac{1}{2}$ in. into the mortar bed.

The speaker has had some experience with the Headley road planer for smoothing down bituminous roads, and finds that it works fairly well when the pavement is soft. It can be adjusted to any desired depth, and in warm weather the surface planed will iron out under traffic. Some of the gentlemen from Philadelphia may be able to give additional information on these matters.

Mr. MacGregor.

R. A. MacGregor,* M. Am. Soc. C. E.—In reference to the specifications for the sizes of broken stone, a definition by the size of the hole in the screen seems to be most practical. In a large part of New York State there are no regular numbered sizes, and even the diameters of the holes in the screens are indefinite. When the size of the hole is specified, there is not much variation in the sizes. That definition is adopted to a large extent, if not entirely, in New York City.

In reference to concrete foundations, the speaker has taken up many during the past few years, and has observed them carefully, but has seen no sign of shattering or failure. He has found broken concrete foundations over sunken trenches, but has never observed which had gone to pieces. The foundations examined were in some cases only 3 or 4 in. thick, and some were old enough to have been made with Rosendale cement, yet they broke in pieces 6 to 12 ft. across and required heavy sledging to break them.

Mr. Vail.

E. M. Vail,† Assoc. M. Am. Soc. C. E.—When there is a shoulder of shifting sand or clay by the side of a hard pavement, the speaker has found that a rut is formed immediately adjoining the pavement. In some roads in New Jersey, a strip, 2 ft. wide and 3 or 4 in. deep, is dug out, filled with crushed stone, and a dressing of light asphalt or tar is applied partly to heal or bind it, so that the hard pavement will not come directly against the soft shoulder.

It has been found that traffic does not grind off or work into a straight rut along the cheaper construction as it does adjoining pavements of the better class.

Mr. Spencer.

W. B. Spencer,‡ Esq.—About 6 years ago, the speaker had a contract with the Pennsylvania State Highway Department for the construction of a bituminous pavement near Harrisburg, Pa. The specifications did not detail any provisions for the shoulders, but, as the roadway was narrow, the speaker thought that they should be protected, especially as he had to guarantee the pavement. He purchased from a near-by quarry a quantity of "one-man" stone, and placed it to a depth of about 8 in. in the shoulders, so that it came up to the top of the pavement. The stones were irregular in shape, and as the hot mix was placed along the edges of the road, it was bound in to the edges of the stones, thus forming a bond between them and the pavement. That furnished an effective and inexpensive shoulder, which is still in good condition.

Mr. Hemstreet.

G. P. Hemstreet,§ Esq.—The speaker would like to emphasize one point, namely, that the penetration at 77°, without taking it at any other temperature, gives very little information. Those who are in practice know that almost any kind of asphalt or bitumen, if it can be

* New York City.

† Plainfield, N. J.

‡ President, Continental Public Works Company, New York City.

§ Supt. of Mfg., Hastings Pavement Co., Hastings-upon-Hudson, N. Y.

maintained at 77°, will produce a good road. If roads were never subjected to cold or hot weather, they would wear very much longer; therefore, when bitumen is tested at 77°, that is a temperature at which almost any kind will give good results. The question is how will it act in cold or hot weather? Some information on this point can be obtained by taking the penetration at different temperatures, perhaps, at 32, 77, and 115° Fahr. Then, if these penetrations are plotted, the resulting curves will permit of a comparison of the different materials. In drawing specifications, if a penetration of 50 points at 77° is called for, a dozen different bitumens will meet that requirement; but when these bitumens are tested at 115° or 32°, they will be found to differ greatly. All have complied with the specification at 77°; some are very poor in cold, and others are very poor in hot weather. The only way to get the proper kind of material, as far as penetration is concerned, is to specify limits at three temperatures. Mr. Hemstreet.

The speaker has been studying the problem of the movement of asphalt blocks under traffic for some years, and believes that it is due principally to one cause. It happens not only in pavements of asphalt blocks, but also on some pavements of wood blocks and vitrified brick. He believes that it is caused almost entirely by the great increase in motor truck traffic. A 10-ton roller exerts a pressure of about 400 lb. per inch of width of tire, and moves at a rate of 3 or 4 miles per hour, thus giving the impression that it is solidifying the pavement so that it can sustain great loads; but motor trucks commonly exert a pressure of 700 lb. per inch of width of tire, and travel at the rate of from 8 to 15, or even 20, miles per hour. Very frequently, the tires are in bad order, so that the wheels bump along the roadway, and this sets up a considerable vibration in the entire pavement, the foundation, and the sub-base. The pavement is vibrating, and, at the same time, there is a great pressure forward, thus creating a new problem for the road engineer.

In a Western city, the speaker recently noticed a wood block pavement in which the rows of blocks were curved in the direction of the traffic. This pavement was laid 2 years ago, and the blocks were in straight lines. Of course, the individual blocks had not changed shape, but the whole pavement had slid, so that, in a distance of only about 18 ft. from the curb to the car track, they were from 3 to 6 in. out of line. In the West there are a number of vitrified brick pavements which have been jarred loose by the traffic, and the bricks are beginning to get out of line. That condition was first noticed in asphalt block pavements about 3 years ago.

In some instances where the blocks had commenced to get out of line, the first attempt to stop them was to take up a row, dig out enough concrete below, and bed in mortar a steel strip, $1\frac{1}{2}$ in. wide and $\frac{3}{16}$ in. thick. The strip was set on edge about $\frac{3}{4}$ in. deep in the mortar,

Mr. Hemstreet.

and ¾ in. projected up between two successive rows of blocks. Where the workmanship was good, this stopped the creeping, as it tied the pavement to the concrete foundation. In practice, however, it was difficult to embed the strip effectively, certain parts being either too high or too low. The final method used to overcome this trouble was to put a steel hoop ½ in. into the block and bed it ½ in. in the mortar. Where this has been done, it has been effective. The blocks containing the steel strips are prepared at the plant and shipped to the street with the others. The anchor blocks are bedded in the mortar at intervals of about 15 ft., and this keeps the pavement from creeping.

Engineers should pay special attention to the necessity for increased foundations on main thoroughfares where there will be heavy motor traffic. A few years ago suburban towns received all their supplies by boat or by train, but, at present, these are conveyed almost entirely by motor trucks. There are now from 5 000 to 6 000 motor trucks running from New York City to outlying suburbs, and delivering supplies of every kind. This condition calls for better foundations for such heavy traffic. It has been the common practice to make these concrete foundations 4 in. thick, but there have been instances where they have been broken into pieces from 6 to 12 in. square, after the pavement had been laid from 5 to 7 years. These changed conditions have been caused by the great increase in motor trucks during the past 2 or 3 years.

Mr. Dallis.

PARK A. DALLIS,* M. AM. SOC. C. E.—In Georgia, the rutting and the waves and ridges are caused principally by the non-draining of the clay sub-strata which retain the water and thus soften the foundation. The light oils, also, will cause the sub-bed to give way and make ruts or waves. Light oil is used on country roads and on fine macadam with good results.

Mr. Tribus.

L. L. TRIBUS,† M. AM. SOC. C. E.—Sometimes the problem is, not to prevent waves in pavements, but how to remedy the trouble. Both conditions must be met. The speaker has in mind two old roads on which the traffic is quite heavy. Each has had a heavy Telford sub-base and macadam surface in place for many years, and each has been re-treated, by penetration methods, with heavy asphaltic oil and fresh broken stone. As the season progressed, each developed a series of waves, almost at right angles to the line of traffic, and almost high enough to make one seasick in traveling along the road.

One road was remedied by heating, picking up, and removing the hummocks and putting on successive light layers of fresh broken stone and rolling them at once. This road has remained in good condition now for about 3 years, with an even coat and no indication of the old waves.

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* Atlanta, Ga.

† New York City.

Mr. Tribus.

The other road would not yield to this treatment, so finally the hummocks were burned off, and the whole roadway was covered with a new 2-in. coat of pre-mixed bituminous material. The latter was of a little harder quality than that generally used, so that the heat of the sun might draw out some of the excess of oil in the under layer, and gradually reduce the almost glassy condition of the top coat as laid. That condition is gradually passing off, but during the first 6 months the footing for horses was very bad.

Mr. Jordan.

LEONARD C. JORDAN,* ASSOC. M. AM. SOC. C. E.—In Brooklyn, the speaker has frequently observed curved asphalt blocks similar to those mentioned by Mr. Boorman. On a street recently examined, on which asphalt blocks had been laid directly on the old macadamized surface, the blocks were curved in one direction on one side of the roadway and in the opposite direction on the other side. They had moved with the traffic, and irrespective of light grades. The courses had been pushed as much as 1½ ft. from the original lines, and the blocks, previously rectangular in shape, had become parallelograms. It is said that the old macadam was dusty when the new surface was laid, therefore, the excessive creeping was probably due largely to the resulting lack of anchorage.

Mr. Durham.

H. W. DURHAM,† M. AM. SOC. C. E.—In European cities, concrete foundations have an average thickness of about 6 in. In London, on the principal thoroughfares, they are 12 in., and, on a small percentage of other streets, 9 in. thick. In Paris, however, only one-eleventh of the stone pavements have any foundation other than sand. In Brussels, the standard thickness is 4 in., and the streets have as good a surface as those in any other European city.

Mr. Headley.

W. T. HEADLEY,‡ ESQ.—The speaker is the originator or inventor of the road planer which has been mentioned. It is made of channel iron, is 10 ft. long, and has three sets of cutting teeth which are set at different elevations. These teeth are of ¾-in. steel, with a square base, and the corners are set vertically. The first row of teeth is set about 1 in. above the roadway to be cut down; the next row is elevated about ½ in.; and the third row is even with the surface. Just behind the teeth there is a steel knife set at an angle, and, after the material is planed loose, this knife scrapes it to one side. The machine was made for the purpose of removing the high spots or tops of the waves in a bituminous roadway, not to remove the whole blanket coat, as some engineers have thought; it is scarcely strong enough for that, though the speaker hopes to design a machine which will meet that requirement.

* Brooklyn, N. Y.

† New York City.

‡ President, Headley Good Roads Company, Philadelphia, Pa.

Mr. Headley.

The machine was made originally to plane down a bituminous road on which considerable trouble was caused by waves and hollows. This road had been surface treated with a very slow-hardening residuum asphalt and covered with stone chips. The road was in good condition during the first season, but when the bitumen began to exude, an application of stone chips was made; the bitumen continued to appear, however, and more and more chips were applied. This, of course, soon produced an excessive thickness, and the surface began to push, and show humps and waves. Then the speaker designed and made the planer, and it was effective in removing all the inequalities in the surface.

Waves in pavements are of several kinds, and are due to many causes. In the early days, when pavements were constructed principally by the penetration method, such waves were generally due to an excess of bitumen and to attempts to absorb it by using screenings. Such screenings generally contained large percentages of dust, and where the proportion of dust was greatest, the waves were most numerous. When clean stone was used, the waves did not occur. Another cause of waves is a loose foundation. In using the planer, the speaker found that, on cutting off the higher parts of some of the waves, the foundation had been pushed up.

Another kind of a wave is caused by the pushing effect of the traffic. By using a cut-back product, which is a thin material for surface treatments, the stones in the surface are cemented together, and no waves occur. A thick mass, however, causes them. The cut-back may be an asphalt or a tar. It will go down into the mass of stone, and after the flux—which has been the medium to carry it in—has evaporated, the whole pavement becomes quite solid. Macadam roads in Philadelphia, treated in this manner, have been quite successful.

In reference to the pressure used in spraying machines, the speaker has noticed that about 30 or 40 lb. per sq. in. is all that is needed. If the material does not go on properly under such a pressure, the plug should be changed for a larger one.

Mr. Connell.

W. H. Connell,* Assoc. M. Am. Soc. C. E.—For the past few years, the speaker has used an asphalt cut-back for superficial bituminous surface treatments. The specifications call for an asphalt having a penetration of from 100 to 125, cut back with a commercial naphtha of from 50 to 54 gravity, mixed in the proportion of 65% asphalt to 35% naphtha. The first treatment has been put on at the rate of about $\frac{1}{3}$ to $\frac{1}{2}$ gal. per sq. yd., and lasts from 1 to 2 years, depending on traffic conditions. The second treatment is applied at the rate of about $\frac{2}{10}$

* Philadelphia, Pa.

gal. per sq. yd. These treatments have been successful for 4 years, and present indications are that they will continue to be successful for superficial bituminous treatments. Mr. Connell.

In inaugurating this treatment, the basic idea was to utilize an asphalt with sufficiently low penetration to enable it to set up somewhat similar to the superficial tar treatments, and also to use a method of applying it so that it would be thin enough to go into the road. It is applied with a pressure distributor, and, of course, the naphtha simply acts as an agent to carry a certain percentage of the asphalt down into the road, and leaves a light crust on the surface similar to a paint coat. The naphtha, of course, evaporates and leaves the asphaltic paint coat. One of the main purposes in applying a light coat is to paint the road so that when it wears out, it will only require repainting and touching up. In this way, we have avoided, and expect to avoid, any waves in the surface similar to those which are very common where the thicker carpet or pad method of treatment is used. When the road needs repainting or re-treating, it is important that it should be done at once, for if it is postponed for 2 or 3 weeks, it is sometimes necessary to re-surface the road. This treatment, of course, is only used on a water-bound road constructed of trap rock, hard limestone, granite, or some other hard stone. If the stone is very soft, similar to the limestone used in Chicago, Ill., and St. Louis, Mo., for road purposes, the superficial treatment would not last.

On page 3007,* there is the following paragraph:

"The use of fine sand on top of bituminous concrete is open to grave objections, and the use of clean stone chips or small gravel free from particles that will pass through a 10-mesh sieve seems preferable."

Gravel is hard, and when the chips pulverize they will be finer than the sand. It is presumed that this paragraph does not refer to the practice of throwing fine sand on top of bituminous concrete after the road is constructed, but merely to the use of such sand in excess. Some explanation of this point seems to be necessary.

The report contains no reference to the bituminous surface treatment, other than the carpet method. This seems to be an oversight, as this is a standard method, used generally, and should be referred to in the report.

C. R. Allen, Jr.,† Assoc. M. Am. Soc. C. E.—In a light surface treatment, a pressure of 50 lb. per sq. in. seems to be excessive, and yet, on a road in which the speaker was interested, that pressure was applied and had the effect of driving the dust away and cleaning the road ahead of the oil application, though such a pressure was not actually required to drive the oil. The results attained fully warranted the use of this high pressure. Mr. Allen.

* *Proceedings*, Am. Soc. C. E., for December, 1914.

† Albany, N. Y.

Mr. Allen. Mr. Ripley has referred to the asphalt being colder when applied than it should be. Provided it can be made to penetrate the stone, the speaker doubts whether it can be too cold. He has seen large quantities spoiled by over-heating, by improper methods of heating, as in small kettles, and by using the last of the material in the kettle, which is often burnt. This has been the cause of the ruin of many roads. Therefore, the question arises: Is it possible to put asphalt on too cold, provided it covers the road uniformly and goes down into the stone properly?

Mr. Ripley has also asked whether he has not made a mistake in getting the asphalt down into the bottom course. The speaker's theory is that the principal function of the asphalt is to hold the stone in place, and that, therefore, its place is in the bottom, and not on top, where it will wrinkle and make waves. The weight and wear of the traffic should come on the stone. Therefore, Mr. Ripley has made no mistake in placing the asphalt in the bottom of the road.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HENRY MARTYN ROOD, M. Am. Soc. C. E.*

DIED DECEMBER 4TH, 1914.

Henry Martyn Rood, through a long line of Colonial ancestors, was descended from the illustrious company of pioneers who, imbued with high religious ideals, and seeking an enlarged freedom of life and conscience, braved the perils of a storm-tossed ocean, an inhospitable wilderness, and a barbarous people, came in the *Mayflower* and, settling in New England, stamped largely on the New World their most virile characteristics of love and zeal for righteousness.

Many years afterward Mr. Rood's father, the Rev. David Rood, inspired with the fervid religious zeal of his ancestors, determined to carry the Gospel of Jesus Christ to the savage tribes of the dark continent of Africa, and so Henry Martyn Rood was born in Amanizimtoti, Natal, South Africa, on February 21st, 1853.

Some of the reminiscences of Mr. Rood's early life were most interesting. From necessity, the companions of his early years were the native Zulu boys of whom he spoke as being highly intelligent, indeed they considered themselves as quite superior to boys of the white race, who required many years of schooling, while they "to the manor born" knew it all instinctively, and felt all the contempt of a dominant race for a weaker and inferior one, and ofttimes jibed the lonely small boy by calling him "white", which was as great a stigma as "nigger" applied to the blacks.

The Rev. Mr. Rood, a graduate of Williams College, and a scholarly man, desiring for his son a liberal education, sent him in 1871 to Phillips Academy, Andover, Mass., from which, in 1873, he entered Yale where he took the regular classical course and was graduated with honors in the Class of 1877.

During his college course, in which he applied himself assiduously to his studies, he became particularly interested in mathematics, and attaining great proficiency in that department, was awarded prizes for superior excellence.

In these college years, his affable disposition and studious habits won for him many staunch and enduring friends. At the conclusion of his college course, he, like many another scholar, soon found he lacked the practical knowledge that would enable him to make a livelihood. Being in ill-health, he returned to the land of his birth and after a time obtained employment with a Government Surveyor.

* Memoir prepared by F. S. Odell, M. Am. Soc. C. E.

Probably Mr. Rood's brief experience at this period as Assistant Land Surveyor (appealing to his fondness for mathematics, and affording healthful occupation for which he had great need), determined his future career, for shortly afterward he returned to the United States and, entering the Rensselaer Polytechnic Institute, completed the engineering course in two years, receiving his degree in 1883. He soon took a position on the Burlington and Missouri Railroad, in Nebraska, and did pioneer work as Transitman and Leveler on location and construction for about one year.

This class of work, with the attendant hardship and exposure, proved too strenuous for the youth, whose health and physical vitality was considerably below the normal. He therefore resigned this position and on his return to the East was for some years associated with the late J. James R. Croes, Past-President, Am. Soc. C. E., as Assistant on surveys for the New Rochelle Water Company's earliest works and on the construction of the Suburban Rapid Transit Company's railway which was then penetrating the "annexed district", now the Borough of The Bronx.

This work included the building of the bridge crossing the Harlem River, at the head of Second Avenue, and extending the line over a right of way purchased and owned by the Company through solidly built-up city blocks. This portion of the road was supported on solid brick piers elevated sufficiently to cross intersecting streets above grade. It was designed for heavy rolling stock, and was operated by steam locomotives far more powerful than any hitherto used on other elevated lines in the city. Incidentally, it may be remarked that this stronger and heavier equipment proved its superiority at the time of the great blizzard of March 13th, 1888, when every other line of transportation, both surface and elevated, in New York City, was blocked with trains everywhere stalled and unable to proceed because of the accumulated masses of driven snow.

After leaving the Suburban Rapid Transit Railway, Mr. Rood was connected for a time with the construction of the second Croton Aqueduct as Engineer for Brown, Howard and Company, Contractors, on a section of the tunnel work. His next work was on the New York State Canals, as Assistant to Ellis B. Noyes, M. Am. Soc. C. E., Division Engineer, who was in charge of improvements on the Champlain Canal, lengthening locks, building bridges, and in various ways increasing the capacity and rendering the waterway more easily navigable.

Mr. Noyes writes, "Mr. Rood, as I recollect him, was of a very retiring disposition, and made few friends outside of those with whom he was brought in contact by his work. Those who did get to know him, liked him very much, and his professional work was always very carefully performed, he being painstaking and careful."

In August, 1891, the appropriation for the work was exhausted, and the engineering party disbanded. Shortly afterward, Mr. Rood became associated with Mr. Purdy, an Architect and Engineer, in Chicago, Ill., where he was engaged in the computations and details of designs of high buildings.

During these migratory years, so well known to perhaps the majority of young engineers, but little time elapsed between successive engagements, due very largely to the ability and fidelity with which he invariably performed his part in each.

About this time (1894) Mr. Rood was married to Miss Grace Sarah Mellen, and located at Mount Vernon, N. Y., where he became Acting City Engineer in the Department of Public Works. Mount Vernon had recently become a city, and, in the many improvements inaugurated then and executed in the ensuing years, the industry, integrity, and ability of Mr. Rood were put to the test and never found wanting.

A political revolution in 1897 resulted in deposing the Commissioner of Public Works, and Mr. Rood lost his position as City Engineer.

In the succeeding years, his services were sought for various municipal works, including water-works for Kingston and White Plains, and paving and sewerage at Port Chester, N. Y., where he had been associated for some time with the writer.

The works of the Municipal Engineer, having to do so largely with sub-surface structures, such as aqueduct tunnels, subways and conduits for electric cables, pipes for sewers, water, gas, and steam, foundations for pavements, piers and abutments, although inconspicuous and unobtrusive, yet minister in the highest degree to the health and well-being, as well as to the convenience and comfort, of modern life, so Mr. Rood, being of an exceedingly modest and unassuming disposition, shunning rather than courting the limelight of popular acclamation, patiently and quietly wrought into the fabric of his life those homely, sterling characteristics and high ideals that make life healthful, helpful, and hopeful, ever radiating an influence for righteousness. He is survived by a widow and five children.

Mr. Rood was elected a Member of the American Society of Civil Engineers on December 3d, 1890.

LAWRENCE CALVIN BRINK, Assoc. M. Am. Soc. C. E.*

DIED MAY 2D, 1914.

Lawrence Calvin Brink was born at Frenchtown, Hunterdon County, N. J., on September 16th, 1869. His parents were Stacey Bray

* Memoir prepared by Jacob Stinman Langthorn and Robert Ridgway, Members, Am. Soc. C. E.

and Henrietta Waterhouse Brink. His father was descended from the early Dutch settlers of New York and New Jersey. His mother's ancestors came from England and settled in New Jersey during the early part of the Eighteenth Century. A great-grandfather, on his mother's side, Joshua B. Calvin, was a descendant of John Calvin, the reformer. Joshua B. Calvin was at one time a member of the Pennsylvania Legislature. Another ancestor on the maternal side was Dr. Benjamin Waterhouse, who was born in Newport, R. I., and is said to have been the physician who introduced vaccination in America. He was a close personal friend of George Washington, and a prominent surgeon in the Continental Army. He taught in Harvard College from 1783 to 1812 as Hersey Professor in Theory and Practice of Physics. Mr. Brink was a nephew of John Waterhouse, M. Am. Soc. C. E., a prominent engineer, who for many years was Chief Engineer of the Manhattan Elevated Railway system in New York City.

After being graduated from High School, at Trenton, N. J., Mr. Brink, in September, 1890, entered Lehigh University, from which he was graduated in 1894 with the degree of Civil Engineer. Before entering college, he had served for a short time in one of the departments of the Pennsylvania Railroad Company.

Soon after his graduation, he became connected with the Engineering Department of the New York Central and Hudson River Railroad Company under Walter Katté, M. Am. Soc. C. E., Chief Engineer, remaining there until he resigned in January, 1900. He was assigned first as Rodman in charge of a line and grade party, on the construction of the new four-track bridge over the Harlem River, at Park Avenue, New York City, and the elevation of the tracks north of it. He entered into this work with the earnestness and enthusiasm which was so characteristic of him, and proved himself to be a young man of unusual ability. His relations with those with whom he was associated there were of the most pleasant character. From October, 1895, to December, 1897, as Junior Assistant Engineer, he was engaged, under the direction of Ezra B. Naylor, M. Am. Soc. C. E., an Assistant Engineer of the Company, on the Park Avenue improvement and on surveys and studies for future improvements. From then until January, 1899, Mr. Brink was engaged on general work on the lines east of Buffalo. Until January, 1900, he was the Assistant Engineer in direct charge of construction, under the Maintenance Department, on the main line between New York City and Albany, and on the Harlem and Putnam Divisions, making surveys, masonry designs for the renewal of bridges, and studies for changes of alignment. During this time, he had direct charge also of dock and yard property in New York City, including the dredging of slips and the extension and reconstruction of piers.

After leaving the New York Central and Hudson River Railroad Company, he was in the employ of Mr. Henry Mesa, Civil Engineer, of New York City, on general surveying and engineering work, until January, 1901.

On January 8th, 1901, Mr. Brink entered the service of the Rapid Transit Railroad Commission, as an Assistant Engineer on the construction of the Rapid Transit Subway in New York City, and was assigned to Section 5-A, which included the work from 41st Street and Park Avenue, through 42d Street, and north on Broadway to 47th Street. For the first two years, he was in charge of laying out this intricate and difficult work, and from that time, until its completion in the latter part of 1904, was in direct charge of the construction of the section. The value of the work under his charge was about $3 500 000, and his handling of the job won the warm commendation of his superiors. On the completion of Section 5-A, he was assigned to other work, and made studies for the ventilation of the subways. In connection with this, he designed the louvres which are a feature of the ventilating system in use to-day.

On January 24th, 1906, he left the service of the Rapid Transit Commission to accept the position of Night Superintendent with the United Engineering and Contracting Company, and in that capacity was engaged on the Company's contract for the construction of the Pennsylvania Tunnels under 32d and 33d Streets, New York City, between Seventh Avenue and the East River. He left the Company's employ in April of the same year.

In the following month, May, 1906, Mr. Brink was appointed Assistant Engineer by the Board of Water Supply, City of New York, and assigned to the Wallkill Division of the Northern Aqueduct Department, with headquarters at New Paltz, Ulster County. He was in direct charge of the studies for the location of the Aqueduct, and in April, 1907, was promoted to the grade of Division Engineer in charge of the Division. The Wallkill Division, as finally located, comprised about 12 miles of the Catskill Aqueduct, which was designed for a daily capacity of 500 000 000 gal. The work included a portion of the grade tunnel through the Shawangunk Mountains, 6¾ miles of cut-and-cover aqueduct on the slope of those mountains and on the east side of the Wallkill Valley, and the deep pressure tunnel under the valley of the Wallkill River. This pressure tunnel, which is 4.4 miles long, was the most important feature of the work. It was constructed from six shafts, ranging in depth from 340 to 480 ft., three of them being circular in section, lined with concrete, and to be maintained as permanent shafts. The tunnel reached a maximum depth below the hydraulic gradient of about 520 ft. The cost of constructing the work on the Wallkill Division was approximately $5 500 000, and it was finished well within the preliminary estimate.

Mr. Brink remained in charge until the work was well advanced, but not until it was completed, resigning in March, 1910, to become General Manager and General Superintendent of the Pittsburg Contracting Company, and continued in that position until his death. Under his direction, the Company completed Contract 52 of the Catskill Aqueduct, in the White Plains Division of the Southern Aqueduct Department. This contract cost about $2 000 000, and comprised about 2.8 miles of cut-and-cover construction, and two difficult grade tunnels in earth, one of them requiring compressed air for its prosecution. He also directed for the Company the work on Contract 65 of the Catskill Aqueduct, one of the four contracts comprising the deep pressure tunnel, 18 miles long, of the City Aqueduct Department. Contract 65 extended from Aqueduct and Burnside Avenues, in the Borough of The Bronx, to Central Park, near Eighth Avenue and West 99th Street, Borough of Manhattan, including Shafts 6 to 12, inclusive, and about 5¼ miles of tunnel. Its estimated cost was more than $5 000 000. Mr. Brink organized the force for this work, and supervised the construction of it until his last illness. At the time of his death, the excavation was nearly completed, and the concrete lining was well advanced. As General Manager of the Company, he enjoyed the distinction of holing through the longest stretch—18 miles—of continuous tunnel in the world. He also managed for the Company a tunnel contract, for the Passaic Valley Sewerage Commission, near Newark, N. J., and assisted in preparing bids for a number of large construction contracts.

In the spring of 1914, he contracted typhoid fever, and, after an illness of five weeks, died on May 2d, 1914.

His long experience on large engineering works in the vicinity of New York City, added to his previous training and his sterling qualities as a man, fitted him for leadership in his Profession, and made him well known and respected among engineers and contractors in and about the City. Mr. Brink was an original thinker, possessing a true scientific mind; he was an earnest and faithful worker, an interesting companion, and a loyal friend. By his human, modest, and lovable qualities, he endeared himself to a large circle of friends, among whom were many of his employees, in whose welfare he always took a warm interest.

He was a Member of the Municipal Engineers of The City of New York and of the Beta Chi Chapter of Phi Gamma Delta Fraternity. He was married, on January 22d, 1896, to Miss Belle Myra Young, of Baltimore County, Md., who survives him.

Mr. Brink was elected an Associate Member of the American Society of Civil Engineers on October 7th, 1908.

HAROLD HANSEN FITTING, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 7TH, 1915.

Harold Hansen Fitting was born on August 21st, 1885, in Grand Rapids, Mich., and moved to California with his parents in October, 1891. He prepared for college at the High School of San Bernardino, Cal., and entered Stanford University in August, 1904, from which he was graduated, with the degree of A. B. in Civil Engineering, in 1909. Prior to entering the University, he was engaged with the Santa Fé Railway on mechanical work in the shops in San Bernardino and, in the course of his studies, both during vacation periods and during a year's absence from the University, he sought and secured work which gave him practical experience in engineering.

In 1906, Mr. Fitting spent the summer as Chainman and Leveler in the employ of the City Engineer of San Bernardino, Cal., and from May, 1907, to August, 1908, he was Draftsman and, later, Assistant Engineer for the Oregon Railroad and Navigation Company on the revaluation of its lines and in charge of parties reporting on the physical condition and preparing estimates for one-half of its mileage.

After his graduation, Mr. Fitting was employed for about one year (1909-10) by the Southern Pacific Company in compiling estimates of revaluation for that Company's lines in Oregon, after which, in 1910, he took charge of the field office of the Stone and Webster Engineering Corporation on the Rubicon hydro-electric development in California.

From November, 1910, until his death, with the exception of only a very few months, Mr. Fitting was with the engineering firm of Duryea, Haehl and Gilman, first as Assistant Engineer and later as Office Engineer. He was entrusted with important problems of engineering investigation and design in power and irrigation work, notably the works of the South San Joaquin Irrigation District, covering about 71 000 acres in the San Joaquin Valley, California, and, during the last year, investigations in connection with the large hydro-electric power project under construction for the Mexican Northern Power Company on the Rio Conchos, State of Chihuahua, Mexico. At the time of his death, he was on temporary leave, being engaged on valuation work for the Oakland, Antioch and Eastern Railway.

On September 20th, 1911, Mr. Fitting was married to Miss Della C. Barnhart who, with a son, survives him. He also leaves a father and two brothers, both the latter being engineers.

On January 6th, 1915, Mr. Fitting was injured in an elevator accident in San Francisco, and died on the day following without

* Memoir prepared by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., a Committee of the San Francisco Association of Members of the American Society of Civil Engineers.

regaining consciousness. The accident resulted in the death of one other passenger and the serious injury of nine more, three of whom were engineers engaged on the valuation work with him.

When Mr. Fitting began his work with Duryea, Haehl and Gilman in 1910, his extraordinary worth and ability was quickly appreciated by the writers, and that appreciation steadily grew. He was a constant inspiration to his associates, not only because of his extreme energy and application to his work, but because he radiated an enthusiasm and good feeling which was felt by all about him.

Mr. Fitting had the ability to get good from his every activity, and was able to apply his knowledge and experience to every situation. Coupled with a fine mind which found real enjoyment in his work, he was an example of the clean-thinking, straightforward man to whom success in his Profession and high standing in the community are assured. Had he lived, he could not have failed to make his impress on the Engineering Profession at large, and his death was a distinct loss, not only to his many friends and associates, but to his Profession as a whole.

Mr. Fitting was elected a Junior of the American Society of Civil Engineers on May 2d, 1911, and an Associate Member on September 3d, 1913. He was also a member of the San Francisco Association of Members of the American Society of Civil Engineers.

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(INSTITUTED 1852)

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TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas.

ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

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TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Robert W. Hunt, George W. Kittredge, Paul M. LaBach, C. G. E. Larsson, William McNab, G. J. Ray, Albert F. Reichmann, F. E. Turneaure, J. E. Willoughby.

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HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....1446 Circle.
CABLE ADDRESS.....“Ceas, New York.”

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS OF THE SOCIETY

April 21st, 1915.—The meeting was called to order at 8.30 P. M.; T. Kennard Thomson, M. Am. Soc. C. E., in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 111 members and 14 guests.

A paper by Adolph F. Meyer, M. Am. Soc. C. E., entitled "Computing Run-Off from Rainfall and Other Physical Data" was presented by the author and illustrated with lantern slides. The author also presented and commented on written discussions by Messrs. Joel D. Justin, A. M. Strong, E. F. Chandler, and T. U. Taylor.

Arthur S. Tuttle, M. Am. Soc. C. E., Chairman of the Committee of Engineers representing National and Local Professional Engineering Societies, reported to the meeting the action of that Committee in reference to the revision of the Constitution of New York State,

stating the principles which should be observed and the recommendations of the Committee to the State Constitutional Convention.

The Assistant Secretary announced the following deaths:

William Harlin Kennedy, of Pittsburgh, Pa., elected Member, September 6th, 1871; died March 14th, 1915.

Maurice Augustus Vielé, of New York City, elected Junior, February 4th, 1891; Associate Member, September 7th, 1892; Member, October 6th, 1897; died April 10th, 1915.

Francis Hopkinson Smith, of New York City, elected Associate, April 5th, 1892; died April 7th, 1915.

Adjourned.

May 5th, 1915.—The meeting was called to order at 8.30 P. M.; Vice-President Gardner S. Williams in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 115 members and 19 guests.

The minutes of the meetings of March 17th and April 7th, 1915, were approved as printed in April, 1915, *Proceedings*.

A paper entitled "Temperature Changes in Mass Concrete", by Charles H. Paul, M. Am. Soc. C. E., and A. B. Mayhew, Assoc. M. Am. Soc. C. E., was presented by the Assistant Secretary, and the subject was discussed by Messrs. Wilson Fitch Smith, George T. Seabury, J. Waldo Smith, T. Kennard Thomson, Gardner S. Williams, and G. Immediato.

The Assistant Secretary announced the following deaths:

John Maxwell Sherrerd, of Pittsburgh, Pa., elected Member, July 10th, 1907; died April 15th, 1915.

George Frederic Simpson, of New York City, elected Member, March 2d, 1887; died April 23d, 1915.

Morton Loudon Tower, of Eureka, Cal., elected Associate Member, May 6th, 1908; Member, July 1st, 1909; died April 15th, 1915.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

May 5th, 1915.—The Board met at 9.30 A. M.; President Marx in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bontecou, Bush, Coleman, Crocker, Davies, Edwards, Fuller, Harwood, Haskell, Hawley, Herschel, Hodge, Keefer, Leonard, Loweth, McDonald, Montfort, Ockerson, Swain, Tuttle, and Williams.

The following appropriations for the work of Committees were made on the recommendation of the Finance Committee:

Special Committee on		Steel Columns and Struts.....	$1 000
"	"	" Materials for Road Construction.	800
"	"	" Valuation of Public Utilities...	800
"	"	" A National Water Law.....	2 000
"	"	" Stresses in Railroad Track.....	4 500
"	"	" Employment and Compensation of Engineers.....	400
"	"	" Floods and Flood Prevention....	500

The following resolution was adopted:

"Resolved: That unless especially authorized by the Board of Direction, it is the sense of this Board that mileage should not be allowed to members of Committees attending meetings held at the same time and place as the meetings of this or other societies, or at cities where expositions are being held."

The payment of $10 000 on the principal of the mortgage of the Society was authorized.

An appropriation of $5 000 to cover necessary repairs and depreciation on the Society House was made, and the expenditure of this amount placed in the hands of the Library Committee.

The matter of the action taken by the Society in regard to the Licensing of Engineers was considered, and the following resolution was adopted:

"Resolved: That the whole matter be referred to letter-ballot of the Society, with a recommendation from the Board of Direction that the previous action in regard to the Licensing of Engineers adopted by the Society in Annual Meeting be rescinded, so that the Board may be free to act in this matter as occasion may demand."

The President was authorized to appoint a Committee of the Board to report upon a revision of the entire Constitution of the Society, such Committee to have the power, in its discretion, of ascertaining the views of the membership on any matter before its incorporation in the proposed revision.

The question of the admission of women to the Society, and all other questions which have been under consideration by the Board and which will require Constitutional amendment, were referred to this Committee.

A Report of the Nominating Committee was received, and the following resolutions adopted:

"Resolved: That it is the sense of this Board that there is no reason why secrecy in regard to the candidates selected should be maintained, but that the Constitutional requirement as to the official notification of the members of the action taken by the Nominating Committee should be carried out as usual; that is, that the list of nominees should not be officially sent out until directly after the first day of October."

A Report was received from the Chairman of the Conference of Presidents of Local Associations held at the time of the last Annual Meeting, together with certain recommendations as to District Organizations in the Society which had been adopted by that Conference, and had been referred to the 14 Local Associations for report. The comments received from these Associations were reported in full.*

This whole matter was referred to the Committee on Revision of the Constitution when appointed.

A letter from General Grenville Mellen Dodge, Hon. M. Am. Soc. C. E., was read to the Board.†

The Constitutional provision in regard to members in arrears for dues was allowed to take effect in the cases of 14 Members, 15 Associate Members, 15 Juniors, and 1 Associate, who had been in arrears for four years.

The resignations of 1 Member and 1 Associate were accepted, the former as taking effect December 31st, 1915.

The Report of Tellers on Membership Ballot canvassed April 7th, 1915, was received, which resulted in the election of 12 Members, 46 Associate Members, 20 Juniors, and the transfer of 16 Juniors to the grade of Associate Member.

Eleven Associate Members and 2 Associates were transferred to the grade of Member.

Applications were considered, and other routine business was transacted.

Adjourned.

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* See page 337.
† See page 331.

SOCIETY ITEMS OF INTEREST

Letter from Gen. Grenville Mellen Dodge, Hon. M. Am. Soc. C. E.

On March 2d, 1915, Gen. Grenville Mellen Dodge was elected an Honorary Member of the Society, and the following letter from him, in reply to the official notification of his election, is printed for the information of the Membership.

"Council Bluffs, Iowa,
"April 19, 1915.

"Chas. Warren Hunt, *Secretary,*
"American Society of Civil Engineers,
"220 West 57th Street, New York City.

"My dear Sir:

"When I received your notice on March 3, 1915, of my election as an Honorary Member of the American Society of Civil Engineers, I was, from sickness, unable to make a proper acknowledgment—could simply return my thanks and acceptance.

"I fully appreciate the great honor conferred upon me, especially as it has the unanimous approval of the Board of Directors of the Society and of all the living Past-Presidents.

"Since the receipt of your letter, I have received many personal letters from members stating that they had recommended my appointment. It was a great gratification to find that many of them knew me in earlier days and had been connected with me, and to see the high positions to which they have risen in their profession.

"Since I was nineteen years old, and until I retired in 1908, I have been actively and continuously on duty in some capacity in my profession, and it has given me an opportunity to see what it has accomplished.

"In the '50's and '60's, my duties were mostly west of the Mississippi River, in exploring the country to Montana, Oregon, California; and in the '70's in Louisiana, Texas, New Mexico, Arizona and Southern California, and I am greatly pleased that Civil Engineers universally made good. I have seen many instances where they lost their lives in contest and fights with the Indians, but I have not known of a case where the Chiefs of the parties did not meet bravely the attacks, and generally successfully, though many very promising young men fell in the fights while performing their duties.

"The Civil Engineers in the Civil War have never been given credit they were entitled to; their brave and valuable services were performed under many disadvantages and discouraging conditions. When the Military Commands were organized and put in the field, there was a great demand for engineers. We had no power to commission them or enlist them. Therefore we had to make details from the different commands, and they were often enlisted men and usually they showed great adaptation for their work and ability and bravery in mapping the country in advance, tracing and making roads, and handling the pioneer corps in their duties.

"They were universally reported for promotion by their superior officers, but for the fact that they were detailed from their companies

and regiments, state authorities refused to promote them in their regiments, and these young engineers of brigades, divisions, and in one case, of corps, were enlisted men but were given commands and were universally recognized. One of the prominent engineers of my corps was an enlisted man of the 2nd Iowa. He was named 'Major', by the men, by which title he was known and acknowledged yet when he left the service at the end of the War, he was mustered out as an enlisted man with an enlisted man's pay, and afterwards became a prominent engineer in building the Pacific roads as well as the Canadian Pacific. Many of these young engineers, after the War, rose to the height of their profession and held high positions.

"After the Civil War, the duties of Civil Engineers greatly changed. Before the War, the Engineer had experience in all the departments of his profession,—in preliminary location, construction, architecture, bridges, shops, permanent structures and terminals. Since the War, all these different duties have fallen to experts, who, by study and practice, have become efficient, and four or five engineers now perform the duties that fell to the engineers in the earlier days, showing a great progress in the profession.

"One of the great satisfactions and pride of my life has been to watch how promptly our engineers tackled and mastered the great problems in developments and internal improvements which these modern times have presented and made necessary. I have often thought if, as a young engineer, I could have had the advantage of belonging to an engineering society and conferring with my comrades and hearing the discussions of any subject that was brought up, what a great benefit it would have been to me.

"But in those days, our duties were far away from any such organizations, and I can appreciate what your Society means to, and as a benefit of, the engineers of our day. And I am glad to see and know what a great Society it is, and how successful it is performing its duties.

"And I wish to express to you and to the Society and all those friends individually who have taken a personal interest in my appointment, my gratitude and appreciation, holding it as I do, one of the greatest honors paid me because it comes from my own profession.

"I am with great respect, cordially,

"Your Obedient Servant,

"GRENVILLE M. DODGE."

New York State Constitutional Convention

On January 6th, 1915, the Board of Direction appointed Messrs. Arthur S. Tuttle, Henry W. Hodge, and Alfred D. Flinn to represent the Society in engineering matters before the New York State Constitutional Convention to be convened on April 15th, 1915.

Acting jointly with representatives from the American Institute of Electrical Engineers, the American Society of Mechanical Engineers, the American Institute of Consulting Engineers, the Municipal Engineers of the City of New York, and the Brooklyn Engineers' Club (and also, informally, with representatives of the American Institute

of Mining Engineers), the following suggestions relative to changes in the State Constitution have been presented to the Convention:

"New York, April 20, 1915.

"To the New York State Constitutional Convention.

"There are few functions of government of greater importance than the administration of Public Works and the regulation of Public Utilities. The former embraces such great undertakings as the new Barge Canal and the State highways. The latter includes the supervision of the corporations engaged in operations intimately connected with the prosperity, health and comfort of all our citizens.

"Under the present Constitution, the way is open for each two-year administration to change completely the management of enterprises which involve operations extending over long terms of years and which can be carried out efficiently and economically only through adhering to well considered plans determined at the outset. Under current practice, changes often are due not to proven incapacity of superseded officials, nor to established merit of their successors, but rather to political expediency. In private business, changes of this character often result in bankruptcy. The principle of continuity was recognized when the Public Service law was enacted, but it has not been written into the Constitution, and until this shall have been done, the method of applying it will be subject to the varying views of the successive Governors and Legislatures.

"Professional engineers are conversant with existing defects in the administration of public works and the regulation of public utilities. In order that the Convention might have the benefit of engineers' views, a public discussion was held in March in which New York members of national engineering societies participated. This has been followed by conferences of committees representing the national societies, and finally by the formation of a Joint Committee including in its membership the national and two local societies. The national societies have about 30,000 members, of whom nearly 6,000 are residents of New York State, while the local societies' membership is about 1,000. The statement of principles and the recommendations transmitted herewith express the views of these committees and we believe that they are entitled to careful consideration by the Convention.

"Since engineering works and organizations are unfamiliar matters to many persons, and have not heretofore been given substantial recognition in our State Constitution, the necessity for our recommendations may not at first be fully apprehended. For this reason, and because of the very large expenditures involved, and on account of the numerous interests affected, we earnestly and disinterestedly urge a very careful study of these governmental functions. Small as well as large investors are vitally concerned. Moreover, state finance and credit are deeply influenced by the wise and economical, or the ill-advised and inefficient administration of public works and regulation of public utilities. We are in accord with a conservative tendency in the Convention, and believe that the Constitution should be as brief as may be and should deal mostly with principles of government, rather than with details of organization, but we are persuaded

that the technical developments of the period which has intervened since the present Constitution was adopted has introduced into government large problems in connection with public works and utilities requiring legislation which should be based on constitutional provisions.

"We trust that these views will have careful consideration and will commend themselves to the Convention. We request opportunities to present before the committees to which these matters may be referred reasons in support of our recommendations. Our services are freely at the disposal of your committees to the end that a structure of efficient and economical State Government may be built on a firm and enduring basis.

"Very respectfully,

"Committee of Engineers:

"Arthur S. Tuttle Henry W. Hodge Alfred D. Flinn	*for American Society of Civil Engineers*
"Gano Dunn Ralph D. Mershon William McClellan	*for American Institute of Electrical Engineers*
"Arthur M. Greene, Jr. Charles Whiting Baker E. Gybbon Spilsbury	*from American Society of Mechanical Engineers*
"Ralph D. Mershon Charles W. Leavitt, Jr. Alten S. Miller	*for American Institute of Consulting Engineers*
"Alfred D. Flinn George W. Tillson Ernest P. Goodrich	*for Municipal Engineers of The City of New York*
"Nelson P. Lewis William W. Brush Jacob S. Langthorn	*for Brooklyn Engineers' Club*

"Arthur S. Tuttle, *Chairman,*
"220 West 57th St., New York.

"Calvin W. Rice, *Secretary.*
"E. J. Mehren, *Assistant Secretary.*

"P. S.—A Committee of the American Institute of Mining Engineers, consisting of W. L. Saunders, Benjamin B. Lawrence, and J. Parke Channing, collaborated in the preparation of the 'principles' and 'recommendations'. Individually, the members of this committee approved them. Pending consideration by the Governors of the Institute, the committee is unable to subscribe officially.

"Principles to be Observed in Amending the Constitution of New York State.

"The Constitution should insure:

"(*a*) A short ballot, making possible close scrutiny of the qualifications of all candidates and concentrated responsibility of elected officers.

"(*b*) Tenure of office for all elected officials, for heads of departments, and for bureau heads, of sufficient duration to attract competent men and to permit them to become increasingly efficient in the discharge of their duties.

"(*c*) A continuing policy in the organization and control of all departments, so that appropriations may be spent most economically and work carried on most efficiently.

"(*d*) Selection of heads of departments by appointment instead of election by popular vote.

"(*e*) Ample opportunity for the proper development of natural resources.

"(*f*) Recognition of the value of technical advice through the inclusion of professional engineers in the membership of Departments or Courts where such advice is essential to the proper conduct of the work or adjudication of the matters involved.

"RECOMMENDATIONS.

"1. The elective office of the State Engineer and Surveyor should be abolished, and the duties should be transferred to the Department of Engineering and Public Works hereinafter proposed.

"2. A Department of Engineering and Public Works should be created to be headed by three commissioners appointed by the Governor, each to have a twelve-year term of office so arranged that a vacancy will be created every four years immediately after the inauguration of a new Governor. Commissioners should be eligible for reappointment. At least one commissioner should be a professional engineer in good standing in his profession, and each should have had experience that would fit him for the duties of the office. At least one and not more than two commissioners should be residents of New York City.

"This Department should have charge of public lands and boundary surveys; of buildings, parks, highways, canals, and other public works, including design, construction, maintenance, and operation; and of the conservation and development of State resources.

"This Department should be divided into suitable bureaus. Each bureau charged with responsibility for engineering work should be headed by a Chief Engineer selected by the commissioners with sole regard to his peculiar fitness for the duties of the bureau.

"3. A Department of Public Utilities should be created to be headed by five commissioners appointed by the Governor, each to have a ten-year term of office so arranged that a vacancy will be created every two years. Commissioners should be eligible for reappointment. At least two Commissioners should be professional engineers in good standing in the profession. Each appointee should have had experience in connection with public utilities which would fit him for the duties of the office. At least two and not more than three commissioners should be residents of New York City.

"This Department should regulate and supervise all common carriers, all water supply, irrigation, drainage, gas, power, lighting, heating, intelligence-transmitting and other public utility corporations operating within the limits of New York State, including similar activities on the part of any other State department or political subdivision of the State.

"This department should be divided into such bureaus as may be

essential. Each engineering bureau should be administered by a Chief Engineer selected with sole regard to his peculiar fitness for the office, and he should have power, subject to the approval of the commissioners, to select and appoint such division engineers as are essential to the proper conduct of his office.

"4. In case provision be made in the Constitution for the creation of one or more departments or commissions charged with responsibility for regulating, supervising, and inspecting buildings and the equipment thereof, trades, mines, industries or labor, each such department or commission should include in its membership at least one professional engineer in good standing in his profession. Commissioners should be appointed by the Governor. Each appointee should have had experience and should possess qualifications fitting him to perform the duties essential to the department or commission.

"5. In case provision be made in the Constitution for the creation of a Court or Board of Claims charged with responsibility for investigating claims against or on behalf of the State, one-third of the membership of such Court or Board should be made up of professional engineers in good standing in the profession, each to have practiced professional engineering for at least ten years and for at least five years to have had responsible charge of important engineering work either as to design or execution.

"6. Removal by the Governor of any commissioner should be made only after the filing of charges and after affording the accused an opportunity to be heard in the matter, provided, however, that at any time within the first six months after making an appointment the Governor may exercise the power of summary removal. A successor to a commissioner who has been removed should be appointed to fill the unexpired term. Vacancies caused by removal or otherwise should be filled under conditions similar to those governing original appointment.

"7. There should be no direct or implied prohibition against legislation which would permit the reference by a court of technical matters made the subject of, or incidental to litigation, to a referee expert in such technical matters, for the purpose of securing a determination concerning the facts, or which will prevent the court from selecting independent expert witnesses or advisers in matters of this character.

"8. In order to make possible the reasonable development of State resources, the present Constitutional prohibition against the use or sale of land and cutting of trees within the limits of the Forest preserve should be removed in so far as such use or sale of land or cutting of trees is essential to such development (Article VII, Section 7), and similarly to permit the development of other natural resources by private enterprise, provision should be made for the condemnation of private property necessary to the construction and operation of works for irrigation, drainage, sanitation, water supply, agriculture, mining, forestry, or power development, through the declaration of such project as *for public use*, subject, however, to the superior right of the State or of a subdivision thereof to condemn the same property or a portion thereof for State or municipal purposes (Article I, Section 7)."

Proposed Meeting Outside of New York City

The Board of Direction has had under advisement the matter of holding one of the regular meetings of the Society at some point outside of New York City at some date between the Annual Meeting and the Annual Convention in September. Communications on this matter have been forwarded to each of the Local Associations of Members, but, inasmuch as there seems to be no particular desire on the part of the membership in any locality to have such a meeting, and also as it is now somewhat late in the season, it has been decided to abandon the idea for this year.

Proposed Reserve Corps of Engineers

General Leonard Wood, U. S. A., some time ago suggested to the Board of Direction that the Society take up the question of the feasibility of the formation of a Reserve Corps of Engineers which would be available in case of emergency, and the Board has appointed Messrs. William Barclay Parsons, of New York City, A. M. Hunt, of San Francisco, Cal., Frank S. Washburn, of Nashville, Tenn., Samuel H. Hedges, of Seattle, Wash., and Charles L. Strobel, of Chicago, Ill., as a Committee to confer with General Wood, to consider and report upon the whole question.

Proposed District Organizations in the Society

At the last Annual Meeting, at the invitation of the Board of Direction, a Conference of the Presidents of the fourteen Local Associations of Members of the Society was held, and that Conference formulated recommendations which were then forwarded to each of those organizations for discussion and report. At a meeting of the Board of Direction, May 5th, 1915, a codification of the replies received was presented. For the information of the membership it may be briefly stated that the replies show that ten of the fourteen Associations, with a total membership of 788, and an estimated membership in the territory covered by these Associations of 1 709, have expressed themselves in favor of the recommendations; and that four Associations, representing a membership of 245, and an estimated membership in the territory covered by these Associations of 589, have expressed themselves as not in favor of the recommendations. Certain reasons given by these Local Associations, both in favor of and against the recommendations, were embodied in the report, and the whole matter was referred to a Special Committee of the Board which is to be appointed to take up the question of a general remodelling of the Constitution of the Society. Undoubtedly, this Committee will ascertain the views of the membership of the Society as a whole on this and on other changes which have been proposed in the present Constitution.

International Engineering Congress, 1915

A circular, entitled "Tour de Luxe to the International Engineering Congress, 1915, and meetings of the American Society of Civil Engineers, American Institute of Mining Engineers, American Society of Mechanical Engineers, and American Institute of Electrical Engineers", has been issued, signed by A. E. Vaughan, and advertising a special tour. Notice is hereby given that this circular has been issued without the authority of the Joint Transportation Committee representing the Societies interested in the International Engineering Congress and in the other meetings to be held in San Francisco in September, 1915, although its form is such that it might easily be mistaken for an official communication.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

June 2d, 1915.—8.30 P. M.—This will be a regular business meeting. A paper by W. L. Du Moulin, Assoc. M. Am. Soc. C. E., entitled "The Pumping Plant of the Morenci Water Company", will be presented for discussion.

This paper was printed in *Proceedings* for April, 1915.

September 1st, 1915.—8.30 P. M.—A regular business meeting will be held, and two papers will be presented for discussion, as follows: "The Twelfth Street Trafficway Viaduct, Kansas City, Missouri", by E. E. Howard, M. Am. Soc. C. E.; and "Pearl Harbor Dry Dock", by H. R. Stanford, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Annual Convention of the Society will be held in San Francisco, Cal., September 16th, 17th, and 18th, 1915, being the Thursday, Friday, and Saturday immediately preceding the International Engineering Congress, which is to be held during the week beginning September 20th.

Three of the other National Engineering Societies, under whose auspices the International Engineering Congress is to be held, will also hold meetings in San Francisco at about that time.

Arrangements have been made for a special train, and possibly more than one train, to accommodate the members of all these Societies who wish to attend their own meeting as well as the Congress.

A general circular containing information as to transportation, hotel rates, etc., has been issued, and later a programme more in detail will be sent out.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed fre-

quently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meetings)

February 19th, 1915.—The meeting was called to order at the Palace Hotel; President Hunt in the chair; E. T. Thurston, Jr., Secretary; and present, also, 71 members and guests.

Mr. J. C. Morrell, an architect representing the Government of Sydney, Australia, was the guest of the Association, and addressed the meeting briefly on architectural and engineering conditions in Australia.

President Hunt announced the death, on January 6th, 1915, of H. H. Fitting, Assoc. M. Am. Soc. C. E., and Mr. H. L. Haehl read a memorial of Mr. Fitting prepared by a Committee of the Association.

A paper by Mr. A. H. Markwart entitled "Building the Exposition", was read by Mr. J. D. Galloway, in Mr. Markwart's absence, and was illustrated with stereopticon views.

Adjourned.

April 16th, 1915.—The meeting was called to order at the Palace Hotel; Vice-President Haehl in the chair; E. T. Thurston, Jr., Secretary; and present, also, 54 members and guests.

The Board of Directors reported the results of the postal card inquiry as to the wishes of members regarding changes in the character of the meetings. The suggestions received were generally discussed by those present.

The Chairman for the Committee appointed to provide entertainment for visiting members during the Annual Convention, reported progress.

The following resolution was referred to the Board of Directors:

"Resolved.—The Secretary, as a part of his duties, is hereby required to endeavor to bring together employers in need of engineering help and engineers requiring employment; the manner of doing this being left to the discretion of the Secretary; and also that the Secretary be allowed an amount per month to help cover clerical expenses in connection with this matter, said amount to be decided by the

Board of Directors, and to be suited to the amount of work the Board may deem it advisable to undertake."

A report relative to the proportion of Association members to Society members within 100 miles of San Francisco, was made by the Secretary, and showed that for less than 50 miles the percentages of Members, Associate Members, Juniors, and Associates, respectively, were 71, 49, 27, and 60, and for more than 50 miles, they were 13, 8, 0, and 0, respectively.

A paper by Mr. H. H. Hail, entitled "The Water Supply for Klondike Hydraulic Mines", was presented by the author who illustrated his remarks with many stereopticon views of interior Alaska and of construction details and methods. A general discussion of the paper followed.

Adjourned.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

April 10th, 1915.—The meeting was called to order at the Albany Hotel; President Vincent in the chair; Roger W. Toll, Secretary; and present, also, 11 members and guests.

The minutes of the meeting of March 13th, 1915, were read and approved.

A letter from Mr. Richard L. Humphrey, President of the Philadelphia Association, regarding the recommendations of the Conference of Presidents of the Local Associations, was read, but no definite action was taken.

A letter from the Colorado Scientific Society offering the Association the use of its auditorium in the new State Museum Building was read.

On motion, duly seconded, the Chair appointed Messrs. W. W. Curtis, Chairman, W. A. Bartlett, and E. C. Van Diest, a committee to report on plans for the reception of Members of the Society passing through Colorado Springs on their way to attend the International Engineering Congress in San Francisco.

Mr. Charles V. Eades addressed the meeting on the subject "Water-Proofing and Insulation". A vote of thanks was tendered Mr. Eades for his paper.

Adjourned.

During the afternoon of April 10th, 1915, and preceding the meeting, a party consisting of members of the Association and their friends visited the Colfax-Lorimer Viaduct now in course of construction. The party was in charge of Mr. H. S. Crocker, Consulting Engineer, and the Assistant Engineers.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. P. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

The proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on January 6th, 1915.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

At the meeting of the Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.), on April 14th, 1915, the following officers were elected for the ensuing year: J. F. Coleman, President; W. B. Gregory and A. M. Shaw, Vice-Presidents; Ole K. Olsen, Treasurer; and E. H. Coleman, Secretary.

Northwestern Association

The proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.) was considered and approved by the Board of Direction of the Society on November 4th, 1914. F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

(Abstract of Minutes of Meeting)

April 27th, 1915.—The meeting was called to order at the Engineers' Club of Philadelphia; President Richard L. Humphrey in the chair; W. L. Stevenson, Secretary; and present, also, about 50 members.

A resolution relative to the appointment of a committee to draft suitable minutes on the death of Mr. William Hunter, was submitted by the Board of Direction, and, on motion, duly seconded, the President was authorized to appoint such a committee.

Announcement was made of the election by the Board of Direction, of Mr. Edgar Marburg, as Vice-President, to fill the unexpired term of Mr. William Hunter, deceased, and of Mr. H. H. Quimby to fill Mr. Marburg's unexpired term. The appointment of the Nominating Committee for officers for 1916, was also announced.

A resolution relative to the affiliation of the Association with the Engineers' Club of Philadelphia, subject to the approval of the Board of Direction of the Society, was submitted by the Board of Direction, and on motion, duly seconded, was approved by the meeting.

Arthur P. Davis, M. Am. Soc. C. E., Director of the U. S. Reclamation Service, and one of three engineers sent by the American Red Cross, in 1914, to China, delivered an illustrated address on "The Flood Problem in China".

On motion, duly seconded, President Humphrey was elected as the Association's representative on the governing body of the Engineers' Club of Philadelphia, subject to the approval, by the Board of Direction of the Society, of the affiliation of the Association with the Engineers' Club.

Letter-ballots were canvassed on the proposed Amendment to the Constitution, providing that the three latest living Past-Presidents who are members of the Association, shall be members of its Board of Direction, and the Amendment was carried unanimously.

As a result of this Amendment, and owing to the death of Mr. William Hunter, the Board of Direction is now as follows: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow and Edgar Marburg; Directors, John Sterling Deans, J. W. Ledoux, H. H. Quimby, and H. S. Smith; Past-President, George S. Webster; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

Adjourned.

Portland, Ore., Association

At the meeting of the Association on October 21st, 1914, the following officers were elected for the ensuing year: President, George C. Mason; First Vice-President, W. S. Turner; Second Vice-President, John T. Whistler; Treasurer, G. B. Hegardt; and Secretary, Charles J. McGonigle.

St. Louis Association

The proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on October 7th, 1914.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensuing year: President, R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

(Abstract of Minutes of Meetings)

March 29th, 1915.—The meeting was called to order at 12.15 P. M., at the College Club; President R. H. Ober in the chair; Carl H. Reeves, Secretary; and present, also, 27 members and guests.

The minutes of the meeting of February 22d, 1915, were read and approved.

Mr. H. F. Tucker addressed the meeting relative to the efforts of the Washington Association of Engineers to unite all practicing engineers on an ethical basis both as to fees and general business practice.

An invitation from the Engineering Open House Committee of the University of Washington to attend an Open House on April 15th, 1915, was read and accepted.

Letters and reports from the Local Associations of Cleveland, Portland, Ore., and Southern California stating their action *in re* the report of the Conference of Presidents of Local Associations held at the Society House on January 19th, 1915, were read.

Letters from Messrs. Daniel Bontecou and C. F. Loweth, relative to Engineering Legislation were read.

A letter and a copy of the Pure Commodities Act, from Dr. H. G. Byers of the University of Washington, were presented, and referred to the Committee on Legislation.

Messrs. M. O. Sylliaasen and B. D. Dean addressed the meeting briefly.

The following Committees were continued: The Committee on Amendments to the Constitution of the Parent Society, consisting of Messrs. S. H. Hedges, Joseph Jacobs, Robert Howes, A. H. Fuller, and John L. Hall; the Committee on Legislation, consisting of Messrs. A. S. Downey, A. H. Fuller, A. H. Dimock, C. E. Fowler, Joseph Jacobs, A. O. Powell, and H. L. Gray; and the Committee on Conference, consisting of Messrs. Robert Howes, John L. Hall, and A. H. Fuller.

Mr. C. C. More described the tests which are to be made on the new warehouse of the Port Commission.

Adjourned.

April 26th, 1915.—The meeting was called to order at 12.15 P. M., at the College Club; Past-President Ernest B. Hussey in the chair; Carl H. Reeves, Secretary; and present, also, 23 members and guests.

The minutes of the meeting of March 29th, 1915, were read and approved.

The resignation of Mr. F. S. Burroughs was accepted.

The Chairman of the Conference Committee reported progress *in re* the work of the Joint Committee handling the preliminary work of the Amalgamated Technical Societies of the city.

Messrs. Henry Gray and Bertram D. Dean addressed the meeting briefly.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 515 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

The proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on March 4th, 1914. Ulysses B. Hough is President.

Texas Association

The proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on December 31st, 1913. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Materials for Road Construction

April 10th, 1915.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard (Secretary).

Mr. Dean presented a progress report of the Sub-committee on New Forms.

The Chairman presented an outline of the work to be accomplished through the medium of new forms and tests pertaining to non-bituminous road materials.

A communication, dated March 6th, 1915, from the Board of Direction relative to a letter from E. W. Stern, M. Am. Soc. C. E., dated January 28th, 1915, a copy of which was enclosed, was presented for consideration, and a tentative report to the Board of Direction was adopted. The Secretary was instructed to forward this report and state that Mr. Stern's letter had been considered carefully at a

meeting at which all members of the Special Committee had been present, and that the report had been adopted unanimously.

The Secretary presented a communication from Mr. George W. Henderson relative to the Hendersonian pavement. The communication was placed on file.

It was decided that the next meeting of the Committee be held at the House of the Society on May 24th, 1915.

Special Committee on Valuation of Public Utilities

April 8th, 9th, and 10th, 1915.—The meetings were held at the House of the Society. Present, F. P. Stearns (Chairman), Charles S. Churchill, William G. Raymond, Henry E. Riggs, Jonathan P. Snow, William J. Wilgus, and Leonard Metcalf (Secretary).

The work done by the members of the Committee was reviewed, and tentative decisions were reached on matters raised in correspondence between the members of the Committee in the interval of time since the last meeting.

The Glossary of Terms was reconsidered in detail and completed, subject to such additions or changes as may hereafter prove necessary.

The Chapters on Property to be Included in Valuation, Original Cost to Date, and Development Expense, were considered in detail and carefully revised or discussed with a view to indicating further revision to be made.

The proposed outlines for certain other chapters were also discussed.

The Committee remained in session continuously for a three days' period, and held one long evening session.

Arrangements were made for future progress, and it was agreed that the next meeting should be held on or about July 6th, 1915, in Boston or vicinity.

Special Committee to Investigate Conditions of Employment of, and Compensation of, Engineers

May 5th, 1915.—The meeting was held at the House of the Society. Present, Nelson P. Lewis (Chairman), S. L. F. Deyo, C. F. Loweth, and George W. Tillson.

Mr. Tillson was elected Secretary *pro tem.*

Mr. Loweth reported that the Board of Direction had allowed the sum of $400 for the expenses of the Committee for 1915.

Mr. Loweth, who at the last meeting of the Committee was requested to formulate a plan for securing information from railway and other corporations concerning the compensation paid to their engineers, reported that he had started with his own company, The Chicago, Milwaukee and St. Paul Railroad, but at once found himself confronted with serious difficulties in attempting to gather information of this kind; he believed that officers of other companies would find the same embarrassment, and that it was unlikely that the Committee would be able to secure information of value from such sources.

The Chairman submitted correspondence with the heads of a number of technical schools to whom he had written to see whether information of the kind the Committee desires was available with

respect to their graduates and former students. The replies indicated that few, if any, of the schools have accurate information of this kind, although they expressed their willingness to assist the Committee in any way that they could.

It was voted that inquiries similar to those already sent out be addressed to men who had become members of the Society since the last inquiry blank was sent out. The Chairman was authorized to try to secure information from such sources as might be available with respect to the salaries or professional incomes of those in other professions, more especially those of the law and medicine.

It was voted that inquiry circulars be addressed to such members of the local engineering societies appearing in the list of those which have extended their privileges to members of the American Society of Civil Engineers who are not also members of this Society.

After considerable discussion as to the necessity of another meeting in the immediate future, the Chairman and Messrs. Deyo and Tillson were appointed a sub-committee to take whatever steps might be necessary to further the work of the Committee, including the employment of such assistance as might be found necessary.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 312 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From April 4th to May 3d, 1915)

DONATIONS*

TUNNELING:

Short and Long Tunnels of Small and Large Section Driven Through Hard and Soft Materials. By Eugene Lauchli. Cloth, 9½ x 6¼ in., illus., 9 + 238 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $3.00.

This treatise on tunneling was prepared, it is stated, to supply practicing engineers and contractors with such information as is necessary to them in tunnel work, and to present to students the underlying principles necessary to the solution of problems to be met in driving tunnels for various purposes. Such subjects as drilling, machinery and equipment, compressor and ventilating plants, explosives and their use, etc., and detailed descriptions of the driving of individual tunnels, have been omitted or only briefly described, it is said, because of the necessity to limit the size of the book; but the subjects discussed or theories developed have been illustrated by examples and figures taken from actual practice. The author states that the book is the result of personal observations and studies, together with data collected from reliable sources, and although some of the chapters have already appeared in technical papers, the text has been entirely rewritten to accord with present-day practice. Cost data for various tunnels constructed in the United States are included in the descriptions of such tunnels. In the case of tunnels driven in foreign countries, however, such data are omitted, owing, it is said, to the impossibility of comparison because of the differences in labor, wages, etc., in this and foreign countries. The Chapter headings are: Introduction; Importance of Geological Surveys in Connection with Tunnel Driving; Headings and Tunnels of Small Cross-Section, Driving Methods; Hauling Methods; Tunnels of Medium Cross-Section; Railroad Tunnels of Moderate Length; Drilling Machinery Used in Connection with Tunnel Work; Long and Deeply Overlaid Tunnels; Method of Handling and Removing Excavated Materials; Tunnel Timbering; Lining Deep Tunnels; Determination of the Rock Temperature in Deeply Overlaid Tunnels; Vitiation of the Atmosphere of Long Tunnels; Ventilation During Construction of Long and Deep Tunnels; Tunnels Driven Through Soft Materials; Pressure Acting on Tunnels Driven Through Soft and Cohesionless Materials; Siphon Tunnels; Tunnels Driven with Compressed Air; Miscellaneous Tunnels; Index.

RAILROAD FIELD MANUAL FOR CIVIL ENGINEERS.

By William G. Raymond, M. Am. Soc. C. E. Morocco, 7¼ x 4¾ in., illus., 7 + 386 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $3.00.

As stated in the title, this Manual is intended principally for use in the field, and, besides tables, is said to contain sufficient explanatory matter relating to common field problems to refresh the memories of engineers who have not had much experience in railroad work, as well as some features not found in similar volumes. It differs from the usual railroad field manual in that the author has divided the degree decimally instead of sexagesimally and has based his tables on that method. He hopes that the plan will appeal to railroad engineers because the use of this new method will, he states, save time and lessen the likelihood of error in computation. Instruments to be used with this method should be graduated to read to hundredths of degrees directly or to 0.6 of a minute. Five-place tables have been adopted, it is said, as representing as high a degree of precision as is warranted by field work, and the author advises engineers to read the explanatory text before using the logarithmic tables. The subject-matter relating to spirals and spiral tables is based, it is stated, on the American Railway Engineering Association's ten-chord spiral. The Manual it is said, can be used on the sexagesimal division of the degree by engineers who do not care to have their instruments changed or to adopt the decimal division for their final records, but the author hopes that the new method will be given a trial and prove a real contribution to the Engineering Profession. The Contents are: Simple Compound and Vertical Curves; The Spiral; Logarithms and Trigonometric Functions; Location Theories and Tables; Estimating and Construction Tables; Turnouts and Crossovers; Azimuth, Latitude, and Time; Tables for Metric Curves; Miscellaneous Tables; Adjustment of Instruments; Sexagesimal Trigonometric Functions.

WHO BUILT THE PANAMA CANAL?

By W. Leon Pepperman. Cloth, 8½ x 5¾ in., illus., 14 + 419 pp. New York, E. P. Dutton & Company, 1915. $2.00.

The author of this book was Chief of the Office of Administration under the Second Isthmian Canal Commission, and as such had charge of all the details of purchase and

* Unless otherwise specified, books in this list have been donated by the publishers.

delivery of materials and supplies, contracts, fiscal affairs, correspondence, engineering reports and plans, etc., for that Commission. After a brief review of what had been accomplished on the Canal previous to 1905 when the Second Commission, known as the "Railroad Men's Commission", took charge, the author shows, it is stated, how the railroad men, under the direction of Theodore P. Shonts, laid the foundations on which the Panama Canal was built. He describes how they rebuilt the Panama Railroad; designed, purchased, and installed the machinery; perfected the plans for the disposal of the spoil; purchased and assembled the materials of construction; cleared Panama and Colon of disease and made them habitable by new water supplies, sewers, etc.; determined the character of labor to be employed; recruited that labor and brought it to the Isthmus and cared for it by building houses, hospitals, and schools, and establishing stores, banks, etc. The book, it is stated, gives the credit due them to the railroad men for what they did and how they did it. The text is illustrated by etchings from Mr. Joseph Pennell's series of Canal pictures, which add to its value historically. The Contents are: The French and the First Commission; Panama in 1905; The Railroad Men Take Charge; The Wallace Affair; Creating a Modern State in the Tropics; Solving the Labor Problem; The Reconstruction of the Panama Railroad; How the Lock Type of Canal Came to be Chosen; Panama in 1907; Human Mosquitoes; M. Bunau-Varilla and the Revolution in Panama; The Canal Itself and What it Means; Appendices: A, The First Trade Ship Through the Panama Canal; B, Farewell to Mr. Stevens; C, Some Letters About Mr. Shonts; D, Some Other Men Who Helped; E, President Roosevelt's Endorsement of the Railroad Men; F, "A Triumph for the Engineering Profession"; Index.

SEWAGE PURIFICATION AND DISPOSAL.

By G. Bertram Kershaw, M. Am. Soc. C. E. (Cambridge Public Health Series.) Cloth, 9 x 5¾ in., illus., 10 + 340 pp. Cambridge, England, University Press, 1915. $3.75. (Donated by the Author.)

As stated in the title, this volume is one of the Cambridge Public Health Series, which series is intended to present the latest scientific and practical information by experts on subjects dealing with the public health, in a useful and handy form and in a manner not too technical. In this volume on sewage purification and disposal, the author who has been the Engineer to the Royal Commission on Sewage Disposal for 16 years, has endeavored, it is stated, to present in concise detail the most recent knowledge relating to the various methods of sewage treatment, sewerage systems, the disposal of various kinds of sewage by different methods, costs, etc. A short bibliography of the subject discussed is given at the end of each chapter. The disposal of sewage by dilution has been omitted by the author, as it has been the subject of a previous volume of the series. The Contents are: Introduction; Conservancy Methods, Composition of Sewages, etc.; Sewage Systems, Variations in Sewage Flow, Storm-Water, etc.; Removal of Suspended Matter; Sludge Disposal; Land Treatment; Contact Beds; Percolating Filters; Sterilisation; Trade Wastes; Subject Index; Index of Authors; Index of Places Cited.

DIRECT-ACTING STEAM PUMPS.

By Frank F. Nickel. Cloth, 9½ x 6¼ in., illus., 10 + 258 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $3.00.

This type of pump has received scant attention, the preface states, from writers on pumping machinery. In recent years it has been replaced by the centrifugal pump, but there are still many conditions, particularly where the exhaust steam is usable, where it is said to be especially useful, and the author has considered it a duty to preserve, in this book, the history of the development of this class of pump, together with the results of his thirty years of experience in its use. The development of the pump, its efficiencies and losses under operation, its various types, and details of its design and operation, etc., are some of the interesting topics discussed in the text. The Appendix contains friction tables of Pennsylvania and California oils in pipes of various sizes. The Contents are: Development of the Direct-Acting Pump; Performance Factors; Classification and Types; Pump End Details; Steam End Details; Service Conditions *vs.* Various Types; Duty; Operation and Adjustment of Direct-Acting Pumps; Appendix; Index.

DONNÉES NUMÉRIQUES DE L'ELECTRICITÉ, MAGNÉTISME ET ELECTROCHIMIE.

Publiées sous le Patronage et avec l'Appui du Ministère du Commerce, de l'Industrie et des Postes et Télégraphes, du Comité des Forges de France (Paris), de la Société des Ingenieurs Civils (Paris), de la Société Internationale des Electriciens (Paris). Par P. Dutoit.

W. C. McC. Lewis, et A. Mahlke. Preface de M. Leblanc. (Tables Annuelles de Constantes et Données Numériques.) Extrait du Volume III, Année 1912. Boards, 11¼ x 8¾ in., 9 pp. + pp. 258-417. Paris, Gauthier-Villars et Cie.; Chicago, University of Chicago Press, 1914. 10 francs. (Donated by the American Commissioners of the International Committee on Annual Tables of Constants.)

As stated in the title, this book is Supplement B of Volume III of the Reports of the International Committee appointed in 1909, by the Seventh Congress of Applied Science, to report on various scientific and technical data. This Supplement contains tables of commercial constants, coefficients, etc., relating to electricity, magnetism, and electro-chemistry, and is intended to serve as a concise medium between the physicist who works out the problems by experiment and the engineer who applies them in his practice. The subject-matter has been compiled by specialists from the laboratories and periodicals of the world, and is published in this separate volume in order that it may be more helpful as an easy reference to those who use such data constantly. The preface states that the data relating to dielectrics, to disruptive potential, and to magnetism, together with certain constants concerned with the disruptive potential and the results of the work of M. Peek, are indispensable to those engaged in high-tension transmission by alternating currents. There are Tables of Contents in French, German, English, and Italian, as well as a short bibliography on electro-chemistry. The Contents are: Electricité, Magnétisme, par L. Mahlke: Conductibilités et Résistances Electriques; Forces Thermo-Electriques; Constantes Diélectriques; Potential Explosif, Rigidité Electrostatique, etc.; Ondes Electriques, Photo-Electricité, etc.; Ferromagnétisme; Hystéresis; Magnéto-Chimie; Rotation Magnétique, Phenomenes de Kerr, de Hall, etc. Electrochimie: I, Conductibilité Electrique des Electrolytes, par P. Dutoit; II, Forces Electromotrices, par W. C. McC. Lewis.

MATERIALS OF CONSTRUCTION:

Their Manufacture, Properties, and Uses. By Adelbert P. Mills, Assoc. M. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., illus., 21 + 682 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $4.50.

The preparation of this volume, it is stated, was undertaken to meet the need for a general textbook covering the manufacture, properties, and uses of the more common materials of engineering construction in a thorough and concise manner. It is not intended, it is said, as an exhaustive treatise of the subject, but as a textbook and reference work of somewhat elementary character. In his treatment of the various classes of materials, the author has prefaced each material or class of materials by a discussion of its ordinary applications in engineering construction; this is followed by a study of its manufacture or natural occurrence, and the subject is concluded by a discussion of the physical and mechanical properties in their relations to its uses. Investigational data are presented in the form of curves or diagrams whenever tabulation can be avoided, and discussions of conflicting empirical data on points admitting of controversy are said to have been reduced to a minimum, the author's aim having been to present the subject-matter in a definite concrete form. Discussion of testing materials has also been omitted except in so far as methods of testing cannot be separated from discussions of the properties of materials as revealed by laboratory tests, as the author believes that the place for such discussion is in a laboratory manual. The Contents are: Part I, The Materials of Masonry Construction: Cementing Materials: Gypsum Plasters; Quicklime; Hydrated Lime; Hydraulic Lime and Grappier Cements; Puzzolan Cements; Slag Cements; Natural Cements; Portland Cement; Concrete; Non-Cementing Masonry Materials: Building Stones and Stone Masonry; Bricks and Other Clay Products. Part II, The Ferrous Metals: Pig Iron; Cast Iron; Malleable Cast Iron; Wrought Iron; Steel; The Special Alloy Steels. Part III, The Non-Ferrous Metals and Alloys and Timber: The Non-Ferrous Metals and Alloys; Timber; Index.

MOTOR TRANSPORTS IN WAR.

By Horace Wyatt. Cloth, 7¼ x 4½ in., illus., 192 pp. London, New York and Toronto, Hodder and Stoughton, 1914. 50 cents. (Donated by George H. Doran Co.)

The author, in his capacity as Editor and Consulting Editor of *Motor Traction*, has studied, it is stated, the question of motor transport in military service from its small beginnings in British army manœuvres many years ago to the present time. It has been, it is said, the motor car, the motor van, and the motor lorry which have rendered possible the rapid transport of troops, food, ammunition, and the rapid con-

veyance of wounded to the hospitals, in the present European war, and, in this book, the author, after a short review of the history of motors and tractors, has given in detail, it is stated, the steps taken by the various Governments to develop motor traction as applied to modern military necessities. The Contents are: Introduction; The Scope of the Motor Vehicle; The Importance of the Military Motor; Trials and Manœuvres; Experiences of Motors in Active Service; Motor Ambulance Work; The Transport of Ammunition and Artillery; Armoured Cars and Other Military Motors; The Provision of Military Motor Transport; A Comparison of National Conditions; British Subsidy Type Motors; Transport Motors of Continental Armies; Emergency Measures on the Outbreak of the War.

ENGINEERING ECONOMICS:

First Principles. By John Charles Lounsbury Fish, M. Am. Soc. C. E. Cloth, 9½ x 6¼ in., illus., 12 + 217 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $2.00.

The design of every engineering structure is judged finally, it is stated, from an economic standpoint, and it is essential, therefore, for the engineer and for all who hold responsible positions in connection with the choice, design, construction, and operation of structures to familiarize themselves with the principles which underlie economic judgment. The author's aim in this book has been to present a working knowledge of the first principles in the economics of structures, and it is also hoped that the book will facilitate the introduction of formal instruction in engineering economics in engineering schools. The subject-matter is divided into five parts, of which Part I is introductory. In Part II the meanings of the terms used and a working knowledge of interest and sinking funds are discussed preparatory to an understanding of Part III which relates to methods of solving problems of economic selection. In this part, the author has included fifteen numerical examples of economic comparison and selection worked out in detail. Part IV contains a bibliography, treatises, and tables relating to the subject and published references to cost data and methods of estimating, and Part V is devoted to tables of formulas and values. The Contents are: Part I, Introductory: The Problem of Economic Selection. Part II, Elements of the Problem of Economic Selection: Interest; Sinking Fund; First Cost; Salvage Value; Elements of Yearly Cost of Service; Estimating. Part III, Solution of the Problem of Economic Selection: Basis of Economic Comparison; Precedure for Economic Selection; Examples of Economic Selection. Part IV, Bibliography and Depreciation and Life Tables. Part V, Tables: Tables of Formulas; Tables of Value; Index.

CONTRACT AND ESTIMATE RECORD BOOK

For Steam and Hot Water Heating. Compiled by Clarence M. Lyman. Cloth, 13 x 9½ in., 7 + 100 pp. New York, David Williams Company, 1914. $2.00.

As described in the title, this book is intended for use by the steamfitter in making up his costs for estimates or contracts for steam or hot water heating. After rules for determining quantities of radiation, piping sizes, standard radiator tappings, list prices, discounts, and net prices, data relating to fittings, expansion tanks, chimney flues, properties of circles, etc., etc., comes the record book proper. It is divided into sections, such as boilers, radiators, valves, union elbows, etc., under each of which is given all the necessary supplies used under such section, divided into columns, those for the costs being double in order that the estimate may be carried in the first and the actual cost in the second, if desired. The contractor or estimator who uses these forms, carefully filling in all the details, will not, it is said, forget or duplicate supplies or figures, and will be sure that his estimate is correct.

ART DE L'INGÉNIEUR ET MÉTALLURGIE:

Résistances des Materiaux et Données Numériques Diverses: Publiées sous le Patronage et avec l'Appui du Ministère des Travaux Publics (Paris), du Comité des Forges de France (Paris), du Groupe Franco-Belge de l'Association Internationale pour l'Essai des Matériaux (Paris), de l'Iron and Steel Institute (Londres), de la Société des Ingenieurs Civils (Paris), de la Société Industrielle de l'Est (Nancy). Par S. L. Archbutt, G. Fieck et W. Hinrichsen, E. Nusbaumer, et A. Portevin. Preface de A. Mesnager. (Tables Annuelles de Constantes et Données Numériques.) Extrait du Volume III, Année 1912. Boards, 11¼ x 8¾ in., 8 pp. + pp. 510-600. Paris, Gauthier-Villars

et Cie.; Chicago, University of Chicago Press, 1914. 10 francs. (Donated by the American Commissioners of the International Committee on Annual Tables of Constants.)

This book is Supplement F of Volume III, issued by an International Committee appointed in 1909, by the Seventh Congress of Applied Science, to report on various scientific and technical data gathered from the laboratories and periodicals of various countries of the world. These data are compiled by specialists on the subjects and are published annually in order, it is stated, that the producer and consumer may keep up with present practice in their particular lines with little study. The subject-matter of this Supplement is limited to Engineering and Metallurgy and includes tables of coefficients, resistances, and various other data pertaining to building materials, textiles, tissues, paper, air and water, bending, friction, refractory materials, fuels, metals and their alloys, etc. At the end of the book is a Table of Contents and an Index in French, German, English, and Italian. The Contents are: Art de l'Ingenieur, par MM. G. Fieck et W. Hinrichsen: I, Constantes Mecaniques (Matériaux de Construction, Textiles et Tissus, Données Diverses); II, Constantes Thermiques (Produits Refractaires, Combustibles). Metallurgie, par MM. S. Archbutt, E. Nusbaumer, et A. Portevin: I, Données Techniques Diverses sur les Metaux et Alliages (Temperature Critique, Solubilité, Densité, Corrosion, etc.); II, Constantes Mecaniques (Essais de Traction, de Compression, de Flexion, de Dureté, etc.); Supplement.

THE ART OF ESTIMATING THE COST OF WORK

With Special Reference to Unstandardized Operations as in Jobbing Shops or Repair Work. By William B. Ferguson. Cloth, 8 x 5¼ in., illus., 97 pp. New York, The Engineering Magazine Co., 1915. $1.00.

The purpose of this book, is to aid in the standarization of estimating the cost of work, with particular attention to the cost of labor on jobbing work or repairs. The subject-matter is the result, it is stated, of the author's experience in preparing and checking estimates for all kinds of ship construction and repair work and the collection and classification of all kinds of actual cost data, embracing work in both Government and private plants and including costs of ordinary day work, piece work, and premium systems of pay. Each plant, it is stated, must collect its own data and must determine a standard estimate for each unit, whether material or labor cost. The author, in this book, then illustrates by examples, descriptions, and references to costs of repairs and alterations to vessels, how an estimator can compare all costs and estimates for new work or repair work with such a standard estimate, and how data can be collected gradually by the plant by which the best possible estimates can be made quickly and accurately for any item of work which may arise. A short bibliography on cost keeping and business organization is given in Appendix II. The Contents are: Introduction; The Art of Estimating; General Methods by Curves or Graphs; Practical Meaning of the Curves; New Construction Work; Repair Work; Classification of Variable Conditions; Cost Data on New Construction; Piece Work Prices; Unstandardized Work: Symbolizing Labor Operations; Needs for Good Estimates; Planning and Estimating by Operations; Preparation and Use of Curves; Estimating Overhead Expense; The Use and Danger of "Rough" Estimates; Developing an Estimating Section; Conclusion; Appendices.

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National Association of Railway Commissioners. Proceedings of the Twenty-Sixth Annual Convention. Held at Washington, D. C., February 17th-20th, 1914. New York, 1915.

Electric Railway Handbook: A Reference Book of Practice Data, Formulas and Tables for the Use of Operators, Engineers, and Students. By Albert S. Richey, Assisted by William C. Greenough. New York and London, 1915.

Soil Fertility and Permanent Agriculture. By Cyril G. Hopkins. New York, Boston, Chicago, and London, 1910.

Municipal Chemistry: A Series of Thirty Lectures by Experts on the Application of the Principles of Chemistry to the City, Delivered at the College of the City of New York, 1910. Edited by Charles Baskerville. New York and London, 1911.

Handbook of Machine Shop Management. By John H. Van Deventer. New York and London, 1915.

Wiring of Finished Buildings: A Practical Treatise, Dealing with the Commercial and the Technical Phases of the Subject, for the Central-Station Man, Electrical Contractor, and Wireman. By Terrell Croft. New York and London, 1915.

General Agricultural Chemistry. By Edwin B. Hart and William E. Tottingham. Madison, 1913.

General Principles of the Method of Least Squares, with Applications. By Dana P. Bartlett. Third Edition. Boston, 1915.

Railroads: Finance and Organization. By William Z. Ripley. New York, London, Bombay, Calcutta, and Madras, 1915.

Valve Gears. By Charles H. Fessenden. New York and London, 1915.

Water Supplies: Their Purification, Filtration, and Sterilization: A Handbook for the Use of Local and Municipal Authorities. By Samuel Rideal and Eric K. Rideal. New York, 1915.

Civil Engineering Types and Devices: A Classified and Illustrated Index of Plant, Construction, Machines, Materials, Means and Methods Adopted and in Use in Civil Engineering Works of Every Class. By T. W. Barber. New York, 1915.

Constant-Voltage Transmission: A Discussion of the Use of Synchronous Motors for Eliminating Variation in Voltage in Electric Power Systems. By Herbert Bristol Dwight. New York and London, 1915.

The Design of Steam Boilers and Pressure Vessels. By George B. Haven and George W. Swett. New York and London, 1915.

Erection and Inspection of Iron and Steel Constructions; Written for the Use of Architects, Engineers and Builders and for Civil Service Candidates for the Position of Inspector of Iron and Steel. By L. M. Bernfeld. New York, 1913.

Structural Design of Warships. By William Hovgaard. New York and London, 1915.

Cape Cod and Its Canal. By J. W. Miller. 1914.

Beton-Kalender, 1915. Taschenbuch für Beton und Eisenbetonbau, sowie die verwandten Fächer. Unter Mitwirkung hervorragender Fachmänner, herausgegeben von der Zeitschrift *Beton u. Eisen.* X Neubearbeiteter Jahrgang. 2 Vol. Berlin, 1914.

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CHASE, CLIFFORD EARL. Supt. and Engr., Okanogan Power & Irrig. Co., Brewster, Wash.	April 7, 1915
COVEY, CHARLES HENRY. Asst. Supt., Fraser, Brace & Co., Room 826, Power Bldg., Montreal, Que., Canada.	April 7, 1915
CRANDALL, LYNN. Asst. Engr., U. S. Geological Survey, 421 Federal Bldg., Salt Lake City, Utah. (Jun.)	Dec. 6, 1910
(Assoc. M.)	April 7, 1915
CURTIS, GEORGE DAVE. First Asst. to Chf. Drainage Engr., Board of Commrs., Everglades Drainage Dist., State of Florida, P. O. Box 200, Tallahassee, Fla.	April 7, 1915
DAVENPORT, ROYAL WILLIAM. Asst. Engr., Land Classification Board, U. S. Geological Survey, Washington, D. C. (Jun.)	Oct. 4, 1910
(Assoc. M.)	April 7, 1915
DESMOND, THOMAS CHARLES. Asst. Engr., U. S. Realty & Impvt. Co., 17 Gramercy Park, New York City.	April 7, 1915
ELLIOTT, JOSEPH ALFRED. Supt. and Mgr., Wheatland Office, Wyoming Development Co., Wheatland, Wyo.	April 7, 1915
FAITOUTE, FREDERIC BUTTERFIELD. Supt. with T. A. Gillespie, 50 Park Pl. (Res., 1770 Concourse), New York City.	April 7, 1915
FARLEY, MARCUS MARTIN. Asst. Engr., Headquarters Dept., Board of Water Supply, City of New York, 22d Floor, Municipal Bldg., New York City. (Jun.)	Sept. 4, 1906
(Assoc. M.)	April 7, 1915
GAUSMANN, ROY WARNER. Section Engr., Board of Water Supply, New York City, Brown Station, N. Y.	Mar. 2, 1915
GRIFFIN, JAMES BIRNEY. Chf. Engr., Abbot Kinney Co., 29 Windward Ave., Venice, Cal.	April 7, 1915
HAW, ELMER PERKINS. Dist. Engr., Technical Section, Dept. of Public Works, Republic of Panama, Care, University Club, Ancon, Canal Zone, Panama.	April 7, 1915
HEED, SAMUEL DARLINGTON. Asst. Engr. of Constr., New York Connecting R. R., 110 East 16th St., New York City.	April 7, 1915
HIRSCH, JOHN GEORGE. Civ. Engr., Bates & Rogers Constr. Co., Chicago, Ill., 276 Hanover St., Milwaukee, Wis. (Jun.)	June 30, 1910
(Assoc. M.)	April 7, 1915
HUNTINGTON, WHITNEY CLARK. Engr. with M. S. Ketchum; Asst. Prof. of Civ. Eng., Univ. of Colorado, 1003 Fifteenth St., Boulder, Colo.	April 7, 1915
HUTCHINS, RALPH. Asst. Engr., Baltimore Sewerage Comm., 1533 Linden Ave. (Res., 5 West Preston St.), Baltimore, Md.	April 7, 1915
JAMES, CLEVELAND ABBE. Res. Engr., L. V. R. R., Third National Bank, Buffalo, N. Y.	April 7, 1915

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
KELLY, WARREN WINFIELD. Care, Chf. Engr., A., T. & S. F. Ry., Los Angeles, Cal.		April 7, 1915
KING, ARTHUR CASWELL. Supt.'s Asst., Springfield Water Dept. (Res., 34 Foster St.), Springfield, Mass.	Jun.	Sept. 4, 1906
	Assoc. M.	April 7, 1915
KNIGHT, GERALD WILSON. Asst. Engr., R. Winthrop Pratt, 1272 Virginia Ave., Lakewood, Ohio.		April 7, 1915
KOOP, LOUIS DIETRICH. Asst. Engr., Dept. of Bridges, 464 East 141st St., New York City.		April 7, 1915
LEARNED, CLYDE EMERSON. Care, Mrs. Spears, R. F. D. No. 1, Massena, N. Y.		April 7, 1915
LEET, CLIFFORD STONE. Land Agt., B. & L. E. R. R., 602 Frick Bldg., Pittsburgh, Pa.		April 7, 1915
LOSH, ALBERT RICHARD. Asst. Engr., Office of State Engr., 1018 Houston St., Manhattan, Kans.		April 7, 1915
NELSON, JABEZ CURRY. Prin. Asst. Engr., Ford, Bacon & Davis, 115 Broadway, Room 1610, New York City.	Jun.	Oct. 1, 1907
	Assoc. M.	April 7, 1915
PORTER, RALPH WALDO. 1448 Coutant St., Cleveland, Ohio.		Dec. 2, 1914
PORZELIUS, ALBERT FREDRICK. Care, Portsmouth, Berkley & Suffolk Water Co., Suffolk, Va.	Jun.	Jan. 7, 1913
	Assoc. M.	April 7, 1915
QUERBACH, EARL. Care, Am. Bridge Co., Ambridge, Pa.	Jun.	Dec. 4, 1906
	Assoc. M.	April 7, 1915
QUICK, RAY STEVENS. Draftsman, Div. of Surveys, Bureau of Eng., 3 Wellington Apartment, Sycamore St., Pittsburgh, Pa.		April 7, 1915
RAY, FRANK OLIVER. Engr., The R. Hardesty Mfg. Co., 1811 Market St., Denver, Colo.		April 7, 1915
SMITH, SHALER GORDON. Care, Cedar Valley Elec. Co., Charles City, Iowa.	Jun.	July 1, 1909
	Assoc. M.	April 7, 1915
STEINMAN, DAVID BERNARD. Special Asst. Engr., New York Connecting R. R., 68 William St., New York City.	Jun.	Mar. 1, 1910
	Assoc. M.	April 7, 1915
STINEMAN, NORMAN MERRITT. Bridge Designer, Eng. Dept., C., M. & St. P. Ry., 4923 Winthrop Ave., Chicago, Ill.		April 7, 1915
STURDY, NATHAN HENRY. Engr. with Trussed Concrete Steel Co., 426 Emerson Pl., Youngstown, Ohio.		April 7, 1915
SWENSSON, OTTO JORDAN. Acting Asst. Engr., Bureau of Highways, Manhattan Borough, Municipal Bldg., New York City (Res., 2 Purser Pl., Yonkers, N. Y.)	Jun.	Nov. 8, 1909
	Assoc. M.	April 7, 1915
THOMPSON, GORDON SAXTON. Instr. in Dept. of Mechanics, Rensselaer Polytechnic Inst.; Advisory Engr., Town of Salem, 693 Second Ave., Troy, N. Y.	Jun.	Sept. 5, 1905
	Assoc. M.	April 7, 1915

ASSOCIATE MEMBERS (*Continued*)

	Date of Membership.
WENTWORTH, GEORGE LANSING. Yonkers Insp., Catskill Aqueduct, Board of Water Supply, New York City, 1 Halcyon Pl., Yonkers, N. Y.	Jun. April 6, 1909 Assoc. M. April 7, 1915
WILLARD, ERNEST CLIFFORD. Engr. in Chg., Appraisal Dept., for Henry L. Gray, 841 Henry Bldg., Seattle, Wash.	April 7, 1915
WRIGHT, FRANCIS HERBERT. (International Eng. Co.), 116 South Michigan Ave., Chicago, Ill.	Nov. 4, 1914

JUNIORS

BEERBOWER, DUMONT. 2901 Sixteenth St., N. W., Washington, D. C.	April 7, 1915
BEOLA, PABLO TOMÁS. Gibara, Cuba	April 7, 1915
BERKEFELD, JOHN WOLFRAM. 227 Bonita Ave., Piedmont, Cal.	April 7, 1915
BISHOP, GUY HERSEY. City Engr., Oelwein, Iowa	Dec. 2, 1914
CONNOLLY, DONALD HILARY. First Lieut., Corps of Engrs., U. S. A., Fort Sam Houston, Tex.	April 7, 1915
CRAWFORD, IVAN CHARLES. Instr. in Civ. Eng., Univ. of Colorado, Boulder, Colo.	Mar. 2, 1915
HARTFORD, FRED DAILEY. Designer and Design Checker for H. S. Crocker, 308 Tramway Bldg., Denver, Colo.	April 7, 1915
HELM, FRANK. Office Engr., A., T. & S. F. Ry., Western Lines, La Junta, Colo.	April 7, 1915
HOLROYD, GEORGE MCINTYRE. Cons. and Estimating Engr. (George McI. Holroyd, Inc.), 23 Douw Bldg., Albany, N. Y.	April 7, 1915
HOLSTLAW, CHARLES HENRY ELLIOTT. Engr. (Bidwell & Holstlaw), 401½ Clematis Ave., West Palm Beach, Fla.	April 7, 1915
INGHAM, EDWIN AMBLER. 514 Central Bldg., Los Angeles, Cal.	Jan. 6, 1915
MURDOCK, ROBERT BRUCE. Crown Point, N. Y.	Jan. 6, 1915
OBERMEYER, WALTER SCOTT. 934 St. James St., Pittsburgh, Pa.	April 7, 1915
RANDOLPH, JAMES ROBBINS. Blacksburg, Va.	Nov. 4, 1914
RHYNUS, CLARENCE PAULDING. San. Bacteriologist, U. S. Public Health Service, Hygienic Laboratory, Washington, D. C.	April 7, 1915
STRACHAN, JOSEPH JOSLIN. 371 Grant Ave., Richmond Hill, N. Y.	April 7, 1915
SUMMERS, RICHARD ELVIN JEWELL. 2102 South 8th St., Tacoma, Wash.	Nov. 4, 1914

JUNIORS (*Continued*)

	Date of Membership.
WHITE, ROY ALLERT. Surv. and Insp., U. S. Army Engr. Corps, Box 72, Louisville, Ky.....	April 7, 1915

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ALLARD, THOMAS THROP. 834 North 24th St., Philadelphia, Pa.

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AUCHINCLOSS, WILLIAM S. Atlantic Highlands, N. J.

AVERILL, FRANK LLOYD. Supt., Library of Congress Bldg. and Grounds, Washington, D. C.

BISSELL, CLINTON TALCOTT. Engr., Committee on Fire Prevention, National Board of Fire Underwriters, 76 William St., New York City.

BONSTOW, THOMAS LACEY. Care, S. Pearson & Son, Ltd., Royal Albert Dock Extension, North Woolwich, London, E., England.

BROWNE, WILLIAM LYON. 380 Elm Ave., Westmount, Que., Canada.

BRUCE, JOHN AUGUSTUS. Cons. Engr. (Bruce & Standeven), 430 Bee Bldg., Omaha, Nebr.

BRYANT, BYRON HARKNESS. Care, W. B. Bryant, 1059 North Shore Ave., Chicago, Ill.

BUDGE, EDWARD BARNARD. Cons. Engr., Chacabuco St. 61, Valparaiso, Chili.

COLLIER, WILLIAM NEVILLE. Superv. Supt. of Constr., U. S. Public Bldgs., Room 205, Post Office Bldg., Kansas City, Mo.

DE WITT, PHILIP HOFFECKER. Contr. and Engr. (S. B. Mutchler & Co.), 30 Church St., Room 402 E, New York City.

DORR, EDGAR SUTTON. Office Engr., Sewer Service, Sewer and Water Div., Public Works Dept., 701 City Hall Annex, Boston, Mass.

GOODWIN, JAMES BOWMAN. Asst. Hydr. Engr., Hydro-Elec. Power Comm. of Ontario, 505 West Marion St., Toronto, Ont., Canada.

HANSEL, CHARLES. Pres., Charles Hansel & Co., Suite 1945, Equitable Bldg., New York City.

HODGE, HARRY SEYMOUR. Univ. of Virginia, Charlottesville, Va.

HOLBROOK, JOHN BYERS. Cons. Engr., 52 Vanderbilt Ave., New York City.

HUBBELL, CLARENCE WILLIAM. Cons. Engr., 2334 Dime Bank Bldg., Detroit, Mich.

JARRETT, EDWIN SETON. Contr. (Jarrett-Chambers Co., Inc.), 30 East 42d St., New York City.

JEWEL, LINDSEY LOUIN. Rosemont Villa, Saranac Lake, N. Y.

JONES, ARTHUR LEWIS. Fernyhurst, Ashstead, Surrey, England.

JUENGST, HENRY FREDERICK. Care, Joplin Water Works Co., Joplin, Mo.

KNIGHT, JOSEPH MARR. Care, *Sir* John Jackson, Ltd., 8 New St., Salisbury, Wiltshire, England.

KNOX, STUART KELSEY. 67 Union St., Montclair, N. J.

MEMBERS (*Continued*)

McGONIGLE, CHARLES JOSEPH. Pres., Western Sheet Metal Works, 13th and Pettygrove Sts., Portland, Ore.

MATAMOROS, LUIS. Ingeniero Consultor, P. O. Box 295, San José, Costa Rica.

MATHEWSON, THOMAS KNIGHT. 1563 North Lake Ave., Pasadena, Cal.

MORDECAI, AUGUSTUS. Cons. and Const. Engr., 1311 Citizens Bldg., Cleveland, Ohio.

MORSE, CHARLES FRANCIS. Asst. Engr., New York State Dept. of Highways, 57 South Clinton St., Poughkeepsie, N. Y.

MORSE, HOWARD SCOTT. Director, Cincinnati Bureau of Municipal Research, 804 Neave Bldg. (Res., 2122 Auburn Ave.), Cincinnati, Ohio.

PERKINS, WILLIAM WARR CASSIDY. Chf. Engr., Dunn Wire-Cut-Lug Brick Co., Conneaut, Ohio.

RICHARDS, ALBERT LENNOX. U. S. Asst. Engr., Care, U. S. Engr. Office, Hannibal, Mo.

ROCKWELL, JAMES VINCENT. Civ. Engr., U. S. N., Navy Yard, Brooklyn, N. Y.

ROUKE, LOUIS KEEGAN. 6 Wayne St., Roxbury, Mass.

SHAW, SUMNER FARNHAM. Apartado No. 38, Guatemala, Guatemala.

SIBERT, WILLIAM LUTHER. Brig.-Gen., U. S. A., Fort Miley, Cal.

STEGNER, CLIFFORD MILTON. Archt. and Engr. (Stegner & Hughes), 1012 Commercial Tribune Bldg., Cincinnati, Ohio.

STEPHENS, CLINTON F. Care, Miss Kate Stephens, 420 West 124th St., New York City.

STONE, EVERETT EDWARD. Member, Public Service Comm., State of Massachusetts, 1 Beacon St., Boston, Mass.

STUART, JOSEPH THOMPSON. Pres., Brann & Stuart Co., Inc., 311 Commercial Trust Bldg., Philadelphia, Pa.

SWAAB, SOLOMON MARK. Cons. Engr., 705 Pennsylvania Bldg., Philadelphia, Pa.

SWIFT, WILLIAM EVERETT. With Ford, Bacon & Davis, 115 Broadway, New York City (Res., Amenia, N. Y.).

THURBER, CLINTON DRAPER. Lt.-Commander, Corps of Civ. Engrs., U. S. N., Care, Bureau of Yards and Docks, Navy Dept., Washington, D. C.

VAN NORDEN, ERNEST MAITLAND. Civ. Engr., New York Edison Co., 130 East 15th St., New York City (Res., 1561 President St., Brooklyn, N. Y.).

WAGNER, SAMUEL TOBIAS. Chf. Engr., P. & R. Ry., Reading Terminal, Philadelphia, Pa.

WALSH, GEORGE SCHERZER. Gen. Supt., Chiriqui Ry., David, Chiriqui, Panama.

WHEELER, WALTER SCOTT. Cons. Engr., 30 East 42d St., New York City.

WOLFE, FRANK CHARLES. 130 West Lanvale St., Baltimore, Md.

WOODBURY, WILLIAM HOOK. Asst. Engr., Valuation Dept., G. N. Ry., St. Paul, Minn.

MEMBERS (*Continued*)

WORTHINGTON, CHARLES. 165 West 71st St., New York City.

YATES, PRESTON KING. Cons. Engr., Room 1045, Equitable Bldg., New York City.

ASSOCIATE MEMBERS

ADAMS, CHARLES ROBERT. 1437 L St., Fresno, Cal.

ALLEN, CHESTER SALISBURY. Concrete Engr., Lockwood, Greene & Co., 60 Federal St., Boston, Mass.

BALLOU, HENRY WELCOME. 735 Grosvenor Bldg., Providence, R. I.

BENEDICT, FARRAND NORTHROP. Engr., Thomas Crimmins Contr. Co., 33 South Maple Ave., East Orange, N. J.

BERNHARD, JOHANNES HELENUS. 52 Broadway, Room 1123, New York City.

BESWICK, JAMES EVERETT. 19 Fort Pl., New Brighton, N. Y.

BILYEU, CHARLES SMITH. New York Representative, Colby & Christie, 165 Broadway, New York City.

BINGHAM, CLARENCE ARMINGER. Gen. Mgr., Town of Norwood, Norwood, Mass.

BROOKS, JOHN NIXON. Douglaston, N. Y.

BURNELL, EUGENE. Room 617, Union Bank Bldg., Winnipeg, Man., Canada.

BURROUGHS, FREDRIC SIDNEY. Care, McClellan & Campion, 141 Broadway, New York City.

BUTLER, MILLARD ANGLE. Div. Engr., G. N. Ry., 508 Broadway, Fargo, N. Dak.

CHAPPELL, CLAUDE EDWARD. 402 South Main St., Charlotte, Mich.

CLOUGH, ALBERT HASKELL. 1036 Mills Bldg., San Francisco, Cal.

COLMAN, JAMES BLAINE THOMAS. Cons. Engr., 702 Kresge Bldg., Detroit, Mich.

COWPER, JOHN WHITFIELD. Pres., The J. W. Cowper Co., Inc., Fidelity Bldg., Buffalo, N. Y.

CRANE, ERNEST BUCHANAN. Asst. Engr., C., M. & St. P. Ry., Seattle, Wash.

CUMMIN, GAYLORD CHURCH. City Mgr., Jackson, Mich.

DAY, ERNEST BUEL. Asst. Engr., Hudson & Man. R. R., 30 Church St. (Res., 435 Fort Washington Ave.), New York City.

DIAMANT, ARTHUR HERBERT. Managing Engr., William Oppenheim, Mongaup Valley, N. Y.

DIXON, DE FOREST HALSTED. Second Vice-Pres., Turner Constr. Co., 11 Broadway, New York City (Res., 71 Gates Ave., Montclair, N. J.).

DRAYTON, NEWBOLD. With E. I. du Pont de Nemours Powder Co., Care, Riverside Club, Penns Grove, N. J.

ELLIOTT, JOHN ARTHUR. Dist. Engr., Oregon State Highway Comm., R. F. D. No. 3, Hood River, Ore.

FEREBEE, JAMES LUMSDEN. Prin. Asst. Engr., Milwaukee Sewerage Comm., City Hall, Milwaukee, Wis.

FRANCIS, WILLIAM. Earlham, Iowa.

ASSOCIATE MEMBERS (*Continued*)

GARDNER, HARRY CARTER. Chf. Engr., John H. Wickersham, 724 North Lime St., Lancaster, Pa.

GOODWIN, IRVING DEAN. Asst. Engr., Pittsburgh-Des Moines Steel Co., 806 Curry Bldg. (Res., 1514 Dormont Ave., Dormont), Pittsburgh, Pa.

GRAVES, GEORGE AUGUSTUS. With Valuation Dept., B. & M. R. R., North Station, Boston (Res., 38 Avon St., Reading), Mass.

GRONER, TRYGVE DANIEL BODTKER. Chf. Engr., D. T. & I. R. R., Springfield, Ohio.

HALE, HERBERT MILLER. Care, Holbrook, Cabot & Rollins Corporation, 52 Vanderbilt Ave., New York City.

HAMILTON, FARRAR PETRIE. Care, Southern Pine Assoc., Interstate Bank Bldg., New Orleans, La.

HAMMOND, LESTER CLARK. Supt., Turner Constr. Co., 312 Prudential Bldg., Buffalo, N. Y.

HATTAN, WILLIAM CARY. Div. Engr., Carolina, Clinchfield & Ohio Ry., Lexington, Va.

HAYES, ANDREW JENKINS. 78 Lake Pl., New Haven, Conn.

HAYS, DONALD SYMINGTON. 644 Everett St., Portland, Ore.

HELLER, JOHN WALTER. Eng. Contr., Post Office Bldg., South Orange, N. J.

HOWES, CYRUS PIERCE. Care, Engr. of Structures, B. & M. R. R., North Station, Boston, Mass.

JACOBS, ROBERT HYDE. Senior Asst. Div. Engr., Public Service Comm., First Dist., 70 East 45th St., New York City.

JONES, PERCY FRANCIS. Care, Modesto Irrig. Dist., Modesto, Cal.

JORDAN, MYRON KENDALL. Draftsman, Kansas City Structural Steel Co., 729 Hamilton Terrace, Kansas City, Mo.

KASSEBAUM, FREDERICK WILLIAM, JR. Care, D. Kassebaum, 4541 North Racine Ave., Chicago, Ill.

KEPPEL, PAUL HENRY. Care, Plant No. 2, Du Pont Powder Co., Penns Grove, N. J.

KINGSCOTT, WALTER JOHN. Box 516, Shreveport, La.

KITTS, JOSEPH ARTHUR. 2803 Forest Ave., Berkeley, Cal.

LEACH, THOMAS. 506 Old Orchard Ave., Notre Dame de Grace, Montreal, Que., Canada.

LUDWIG, JULIUS ALFRED. Supt., Bureau of Bldgs., Borough of Manhattan, 20th Floor, Municipal Bldg., New York City.

MCCLAIN, JAMES BROWNSON. Res. Engr., Bridge Dept., Seaboard A. L. Ry., 344½ East Duval St., Jacksonville, Fla.

MASHBURN, LEON WADDELL. Chf. Engr., Southern Eng. Co., Clarksdale, Miss.

MUNN, ALEXANDER MAJORS. Director and Secy., Munn-Reise Constr. Co., Daniels Bldg., Kansas City, Mo.

PETERSON, OTTO WALLACE. Res. Constr. Engr., Modesto Irrig. Dist., La Grange, Cal.

ASSOCIATE MEMBERS (*Continued*)

PHILIPS, GEORGE WASHINGTON. Res. Engr., San. Sewerage Constr., Troy, Pa.

PHIPPS, THOMAS ELMER. Chf. Engr., Public Service Comm., 3444 Florence Court, Seattle, Wash.

PRICE, WILLIAM EDMUND. Gen. Contr., Associated with A. O. Campbell, 610 Majestic Bldg., Oklahoma, Okla.

RABINOWITZ, LOUIS. Care, Barrow Farm, Napanoch, N. Y.

RHODES, CLAUDE IRVIN. Asst. Engr., California State R. R. Comm., 280 Ninth Ave., San Francisco, Cal.

RICE, JOHN MARIE THOMAS. With Morris Knowles, Cons. Engr., 2541 Oliver Bldg. (Res., 5435 Black St.), Pittsburgh, Pa.

RICHARDSON, CHARLES POTTER. Asst. Engr., Track Elev., Rock Island Lines, 7330 Lowe Ave., Chicago, Ill.

RICHMOND, JACKSON LITTON. Gen. Contr., Grant, N. Y.

ROSENTHAL, JOSEPH JACOB. 525 Market St., Room 220, San Francisco, Cal.

SCHEIDENHELM, FREDERICK WILLIAM. Cons. Engr., Benedum Trees Bldg., Pittsburgh, Pa.

SCHRYVER, HOWARD FRENCH. Asst. Valuation Engr., T. & O. Cent. Ry., 405 New York Central Bldg., Columbus, Ohio.

SLAYTON, CHARLES ALBERT. Care, Corrigan & Halpin, Reserve Bank Bldg., Kansas City, Mo.

SMITH, JONATHAN RHODES. Care, W. R. Smith, Merry Hill, N. C.

STRATE, THOMAS HENRY. 3416 Aldrich Ave., South, Minneapolis, Minn.

THOMPSON, MORRIS. Engr. and Gen. Supt., Bates & Hudnall, 561 Grove St., Milwaukee, Wis.

TOLLES, FRANK CLIFTON. 59 Franklin St., Buffalo, N. Y.

TREVARTHEN, DWIGHT CUTLER. P. O. Box 49, Florence, Ala.

WAGNER, ALLAN JOHN. Care, California Highway Comm., 515 Forum Bldg., Sacramento, Cal.

WAIT, BERTRAND HINMAN. Div. Engr., State Comm. of Highways, Columbus Inst., Poughkeepsie, N. Y.

WALTER, FRANK EDGAR. 1030 West Exchange St., Akron, Ohio.

WEIDEL, JOSEPH. 517 West 14th St., Topeka, Kans.

WELLER, WILLIAM EARL. Deputy City Engr., 501 Rugby Rd., Schenectady, N. Y.

WICKLINE, GEORGE GROVER. Care, City Engr., Dallas, Tex.

ASSOCIATE

WILSON, HUGH MONROE. Vice-Pres. and Gen. Mgr., McGraw Pub. Co., Inc., 239 West 39th St., New York City (Res., Little Woods, Park Rd., Hartsdale, N. Y.).

JUNIORS

BOLIN, HARRY WILLIAM. 1620 Jordan St., Portland, Ore.

BOWEN, EDWARD WITHERS. Care, C. W. Requarth Co., Commercial National Bank Bldg., Charlotte, N. C.

JUNIORS (*Continued*)

CARTWRIGHT, HENRY HART. Asst. Engr., Valuation Dept., N. C. & St. L. Ry., Nashville, Tenn.

CLIFT, WILLIAM BROOKS. Secy.-Treas., Capitol City Constr. Co., 1 Berry Blk., Nashville, Tenn.

COCHRAN, JEROME. 1518 Hamilton St., Houston, Tex.

DAVIS, WILLIAM EILERT. 659 Second Ave., San Francisco, Cal.

DENSLER, FRANK HASKELL. 825 Myrtle Ave., Albany, N. Y.

DIMMLER, CHARLES LOUIS. Asst. Engr., Div. of Works, Panama-Pacific International Exposition, 1911 Cedar St., Berkeley, Cal.

GIBBLE, ISAAC OBERHOLZER. With The Trussed Concrete Steel Co., 817 Commerce Bldg., St. Paul, Minn.

HARDING, EDWARD CRITTENDEN, JR. Chf. of Grade Party, New Sewer System, 308 Odd Fellows Temple, Cincinnati, Ohio.

HAVENS, WILLIAM WESTERFIELD. Junior Engr., 7th Div., Public Service Comm., First Dist., New York State, 2231 Grand Boulevard and Concourse, New York City.

HELLING, HARRY ALBERTUS. Asst. Engr., State Highway Dept., Box 742, Liberty, N. Y.

HOSSACK, ARCHIBALD BABCOCK. Solicitor, The American Appraisal Co., 85 Devonshire St., Room 1011, Boston, Mass.

KRAUSS, JOHN JACOB. 992 Warren Ave., West, Detroit, Mich.

LATIMER, CLAUDE ALFRED. 3021 Richmond Terrace, Mariners Harbor, N. Y.

LEMCKE, KARL WOLFGANG. 117 Pine St., Harrisburg, Pa.

LEONARD, EDWARD PHILIP. Care, The Elec. Bond & Share Co., 71 Broadway, New York City (Res., 887 Lincoln Pl., Brooklyn, N. Y.).

MAY, DONALD CURTIS. Care, Gardner S. Williams, Ann Arbor, Mich.

PARKER, HENRY BRACKETTE. Delmar, N. Y.

PERRINE, HAROLD. Care, E. I. du Pont de Nemours Powder Co., Wilmington, Del.

PRICE, JOSEPH. Asst. Supt., U. S. Lighthouse Service, U. S. Lighthouse Office, Buffalo, N. Y.

SEARIGHT, GEORGE PETER. 313 South Hanover St., Carlisle, Pa.

SEE, RUSSELL ALVA. Care, U. S. Reclamation Service, Camp No. 9, Babb, Mont.

SEELEY, HENRY ARTHUR. 130 East 15th St., Room 540, New York City.

SELBY, OSCAR ELLSWORTH. Prin. Asst. Engr., C., C., C. & St. L. Ry., Cincinnati, Ohio.

STARR, WILLIAM H. Hotel Marlton, 3 West 8th St., New York City.

SWARTZ, LEON EMERSON. Care, Bell-Bockel Co., Inc., Altoona, Pa.

TATE, ROBERT L'HOMMEDIEU. 1097 Elmwood Ave., Buffalo, N. Y.

THACKWELL, HENRY LAWRENCE. Civ. and Hydr. Engr., 1345 Franklin St., Denver, Colo.

WEITZNER, HENRY MITCHEL. With New York Municipal Ry. Corporation, 85 Clinton St., Brooklyn, N. Y. (Res., 867 Beck St., New York City).

WELLES, THEODORE LADD, JR. 1427 North Front St., Harrisburg, Pa.

DEATHS

KENNEDY, WILLIAM HARLIN. Elected Member, September 6th, 1871; died March 14th, 1915.

SHERRERD, JOHN MAXWELL. Elected Member, July 10th, 1907; died April 15th, 1915.

SIMPSON, GEORGE FREDERIC. Elected Member, March 2d, 1887; died April 23d, 1915.

SMITH, FRANCIS HOPKINSON. Elected Associate, April 5th, 1892; died April 7th, 1915.

TOWER, MORTON LOUDON. Elected Associate Member, May 6th, 1908; Member, July 1st, 1909; died April 15th, 1915.

VIELÉ, MAURICE AUGUSTUS. Elected Junior, February 4th, 1891; Associate Member, September 7th, 1892; Member, October 6th, 1897; died April 10th, 1915.

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In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
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(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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THE TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MISSOURI

By E. E. Howard, M. Am. Soc. C. E.

To be Presented September 1st, 1915.

Synopsis.

This paper describes the planning, the design, the equipment for, and methods of, construction, of a double-deck, reinforced concrete, trafficway viaduct, 2 300 ft. long, 60 ft. wide, and 120 ft. high, recently opened for traffic in Kansas City, Mo. Some special applications of general theories involved in the calculations for design are given, and summarized and classified costs are included. The features deemed of particular interest are the design of a large arch forming a portion of the structure, and the arrangement of the contractor's plant and equipment.

For convenience the paper is divided into the following parts:

1.—Introductory and historical.
2.—General description of structure.
3.—Loadings, specifications for, and methods of, design.
   A. Investigation of columns.
   B. Design of arch.
4.—Specifications for materials and workmanship.
5.—Equipment for, and methods of, construction.
6.—Quantities and costs.
7.—Personnel.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The description of features and details which may be grasped readily from an inspection of the accompanying illustrations is omitted.

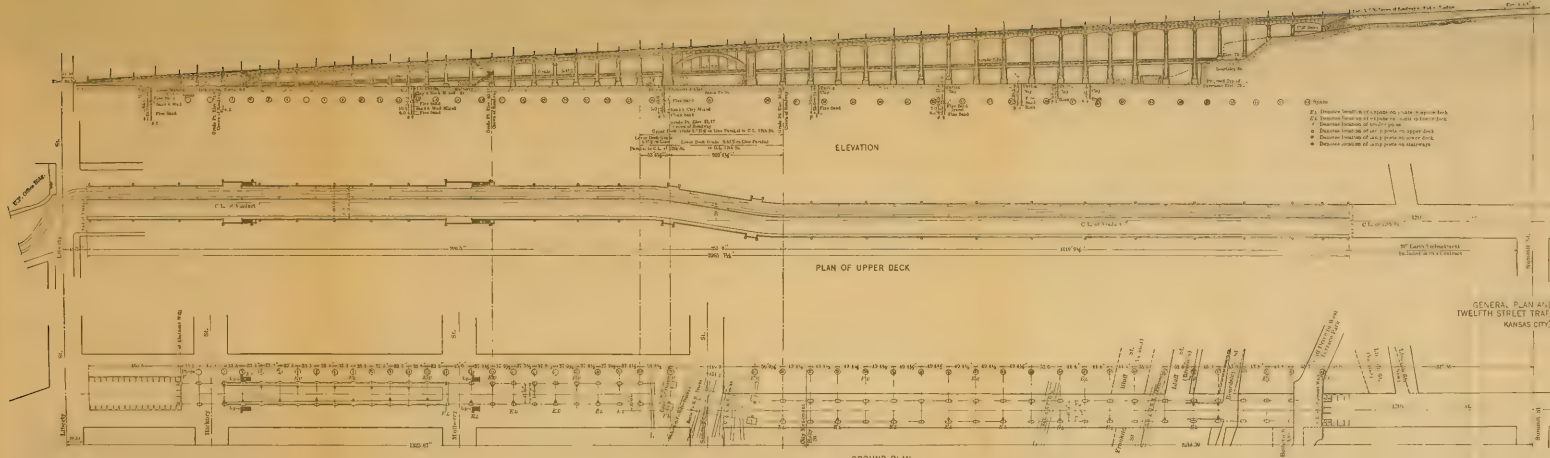
1.—Introductory and Historical.

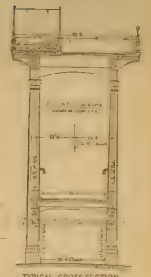
The Twelfth Street Trafficway Viaduct forms the principal part of the Twelfth Street Trafficway, a general municipal improvement designed to facilitate usual street traffic between sections of Kansas City, Mo., which are separated topographically. The total cost of the whole improvement, including land damages, was about $1 000 000, the viaduct structure costing about $650 000. The funds were provided by a portion of a general bond issue of $475 000, by assessments against a benefit district of special tax bills of $325 000, and by a contribution of $200 000 from the local street-car company in return for operating privileges over the structure.

The rather flat, level valley of the Kaw River, in Kansas City, locally known as the "West Bottoms", is about 1 mile wide, and is occupied by railroad yards, freight stations, warehouses, packing-houses, wholesale and manufacturing concerns, and the large stock yards interests, including a considerable percentage of the city's business. It lies at an elevation some 150 ft. below the principal residential, retail, and office building districts of the city, the break between the two elevations being rather precipitous bluffs and hills from 100 to 200 ft. high. Twelfth Street is the principal east-and-west thoroughfare of the city, and may be said to be a central axis, both with respect to the up-town part of the city, and the industries in the Bottoms. There have been only three routes for vehicular travel between these levels: the northern routes, requiring a diversion of about ½ mile north of Twelfth Street, and a southern route, requiring a similar diversion of more than 1 mile south. About thirty years ago, a steel viaduct for cable cars was built on Twelfth Street with a grade of about 13%, and car service was maintained until the present construction was commenced.

A roadway suitable for vehicular, as well as street-car, traffic on a viaduct to the top of the hill, on a viaduct to the bluffs, and in a tunnel emerging at some convenient point, and other such arrangements, have been advocated for many years, and have been made a feature of more than one political campaign, and of franchise negotiations with street-railway interests. The improvement finally realized as the Trafficway

PLATE X
PAPERS, AM. SOC. C. E.
MAY, 1915
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT
KANSAS CITY, MO




TYPICAL CROSS-SECTION

GENERAL PLAN AND ELEVATION
TWELFTH STREET TRAFFICWAY VIADUCT,
KANSAS CITY, MO

extends from Liberty Street to Broadway, a distance of $\frac{3}{4}$ mile, of which the viaduct occupies about 2 300 ft. The remaining portion consists of earth embankments and cuts with the usual street improvements on Twelfth Street, and the regrading of certain side streets to intersect its new grade. A street 60 ft. wide is thus provided on a continuous grade of about 5.5% for a distance of 3 500 ft., substantially from end to end of the improvement. It was physically impossible, without involving prohibitory expense and damage to existing structures, to extend the end of the grade in either direction, so the gradient practically fixed itself. To accommodate traffic desiring a less steep grade, and willing, for such advantage, to travel by a less direct route, a roadway on a lower deck is provided, with a grade of about 2.5 per cent. This lower roadway begins one block east of the upper one and extends to the bluff, a distance of 1 800 ft.; thus enabling all classes of traffic to avoid the grade crossings over the heavy service railway tracks on the street below. Ascending roadways or streets alongside the bluff from the end of the lower deck will terminate in streets at the summit of the hill, giving easy grades, although by indirect routes for team traffic bound up town. These connecting streets are not yet built.

Preliminary studies and estimates were made for viaducts of steel and of reinforced concrete; in the latter case, structures of arch spans, and also of girder spans, were considered. The city officials were favorable to the engineers' recommendations of concrete, and the girder type of structure was selected, the final general plan being as shown on Plate X. The suspended lower deck over Santa Fé Street, of the final plan, was developed instead of short spans with intermediate columns, to meet the desire of the railways for a clear span the full width of the street.

2.—General Description of Structure.

The Twelfth Street Trafficway Viaduct is a reinforced concrete structure consisting of girder spans supported by columns founded in part on rock, in part on soil, and in the main part on concrete piles. Over eight railway tracks in Santa Fé Street a long span is provided by the construction of an arch. The structure is about 2 300 ft. long, 60 ft. wide, and 120 ft. high at its highest point. The columns are in pairs transversely, and are placed so that the upper deck, which has a total width of 60 ft., cantilevers beyond the columns on each side, and

the lower deck roadway passes between them. The upper deck provides a roadway 30 ft. wide paved with creosoted blocks, a sidewalk 5 ft. wide, and a street-car space 22 ft. wide. The roadway is separated from the street-car space by a concrete curb, and on each side of the structure there is a concrete hand-rail. The two street-car tracks are built with the usual wooden cross-ties set in ballast, so that no other traffic can use the area which they occupy. This arrangement, somewhat uneconomical from a highway standpoint, was specified by the city authorities. Iron trolley poles on each side of the street-car space support the usual overhead trolleys. Stairways are provided at Hickory and Mulberry Streets to give access for pedestrians to the roadway from the ground surface. The lower deck, beginning at Hickory Street and ending at Beardsley Street, provides a single roadway, 30 ft. wide, paved with creosoted blocks. The longitudinal girders supporting this deck extend far enough above the roadway to form side barriers.

The upper deck comprises forty-five deck-girder spans of two girders each, varying in length from 33 to about 56 ft., the arch span, and two earth-filled approaches. The lower deck comprises twenty-seven through-girder spans of two girders each, supported on the same columns that carry the upper deck, the suspended deck of the arch span, and the earth-filled approaches. The floor-slabs are supported on cross-girders and cantilever beams. For the crossings of both decks over the streets, near the west end, very shallow floors are necessary, and concrete-encased steel beams in the floors, and concrete-covered steel girders, are used instead of the ordinary reinforcement. Both upper and lower roadways are lighted with incandescent electric lights, placed above the hand-rail for the upper deck, and on brackets on columns for the lower deck.

Below the structure Twelfth Street is about level from Santa Fé Street to the hillside, and gives access to Bluff Street leading northward along the bottom of the bluff. The space transversely between columns at the street level is 30 ft. Near the bluff, passing under the lower deck, the Kansas City Terminal Railway Company has two tracks carried by a concrete girder span above the ground surface of Twelfth Street, making four traffic levels at this point. Higher up on the hillside, just in front of the east abutment, the Kersey Coates Drive of the Park System passes below the upper-deck level.

Twelfth Street from Broadway to Santa Fé Street is 60 ft. wide, but for the three blocks, from Santa Fé Street to Liberty Street, it is only 30 ft. wide. Eleventh Street for these three blocks is 60 ft. wide, and the blocks between Twelfth Street and Eleventh Street are only 40 ft. wide. By securing these three very narrow blocks of property, a single street, 130 ft. wide for a distance of three blocks, was obtained, and the viaduct was placed about midway between, affording an excellent approach and retaining both Twelfth Street and Eleventh Street. This caused a shift in the alignment of about 30 ft., made by two easy curves at Santa Fé Street.

Careful attention was given to the architectural treatment in an effort to secure something more than a plain series of posts and beams. The treatment developed considers the columns as columns with plinths and capitals, and not merely as posts. The bottoms of the upper-deck girders are curved to give an arched or high cambered appearance. The lower-deck girders are straight and in effect are supported by secondary pilasters set out from the main columns. Certain limiting conditions of the railway and street locations crossing under the structure practically determined certain column locations, but the span lengths were made smaller near the lower end of the structure, affording uniformity of appearance and preserving the unity of effect, in spite of the great variation in height of the different columns.

The large arch at Santa Fé Street provides a central feature for the structure and was thus developed. The piers of the arch are carried up to the upper roadway and there support, on cantilevers, bays projecting beyond the usual hand-rail line. The capitals, plinths, mouldings, the projecting panels of the columns, and such features, are bold and heavy, without excess of fine lines, in keeping with the general dimensions of the structure. The architectural appearance has been favorably commented on, and it is believed effects a satisfactory solution of a rather difficult problem of a straight-line structure to be built with all possible economy and without any extra funds for adornment.

3.—Loadings, Specifications for, and Methods of, Design.

Assumed Live Loads.—The following loadings were assumed for design:

1.—For Slabs and Floor Systems:

On street-car tracks, Class 25, consisting of two coupled cars, each 52 ft. long and each weighing 50 tons, with impact allowance loads consisting of percentages of the preceding static loads, determined by the formula,

$$I = \frac{200}{L + 270},$$

where L is the length of track loaded;

On roadways, Class A, with an added impact allowance percentage determined by the formula,

$$I = \frac{100}{L + 150},$$

which, in total, consists of a load varying from 147 lb. per sq. ft., for a loaded length of 100 ft., to 200 lb. per sq. ft.;

On the sidewalk, Class B and impact, a corresponding loading varying from 123 to 167 lb. per sq. ft.

2.—For Girders and Arch Ribs:

On street-car tracks, Class 20, consisting of two coupled 40-ton cars, with impact percentage added;

On the roadways, Class B with impact;

On the sidewalk, Class C with impact, consisting of a load varying from 98 to 134 lb. per sq. ft.

The roadways were also considered to carry a 15-ton road roller, but without impact.

The wind loads figured were 30 lb. per sq. ft. of exposed surface for the unloaded structure, and 15 lb. per sq. ft. of exposed surface for the loaded structure. When considering maximum combined conditions of wind and temperature stresses, the allowable stresses were increased 30%, or as described later.

Allowed Unit Stresses.—For these assumed loadings, the following unit stresses were allowed in the design:

"For steel in tension, 15 000 lb. per sq. in.; for concrete in bending compression, 600 lb. per sq. in., tension zero; for concrete in direct compression, 450 lb. per sq. in., with not more than 400 lb. per sq. in. under expansion plates; for concrete in direct shear, 150 lb. per sq. in.; for concrete in diagonal tension, 35 lb. per sq. in.; for concrete in diagonal tension with part of steel bent up, 50 lb. per sq. in.; for concrete in diagonal tension with full shear reinforcement, 100 lb. per sq. in.; adhesion between concrete and deformed bar, 100 lb. per sq. in."

Designing Rules.—Some of the general rules for proportioning were:

"Column reinforcement shall have hoops spaced not farther apart than fifteen diameters of the vertical bars. The vertical steel shall be not less than 0.8%, nor more than 2.0%, of the minimum section of the column. The ratio of moduli of elasticity shall be taken as 15. The span length shall be taken as the distance from center to center of supports, but not to exceed the clear openings plus the depth of the slab or beam. The width of T-beams shall be considered to be not more than one-fourth of the span. Tension bars shall be spaced apart not less than three diameters of the bar, nor more than the thickness of the slab. If bond stress exceeds 100 lb. per sq. in., the ends of the bars shall be bent into a hook. The distance of the center line of bars to the surface of the concrete shall be not less than 1.3 d plus 0.17 in.; or, say, $\frac{3}{4}$ in. for $\frac{3}{8}$-in. bars, 1 in. for $\frac{1}{2}$ and $\frac{5}{8}$-in. bars, $1\frac{1}{2}$ in. for $\frac{7}{8}$ and 1-in. bars, and $1\frac{3}{4}$ in. for $1\frac{1}{8}$ and $1\frac{1}{4}$-in. bars. When stirrups are bent around the bars, allow an extra distance equal to the thickness of the stirrup, when the above distance is 1 in. or less, and equal to one-half the thickness when the above distance is more than 1 inch."

Controlling Conditions and General Methods.—The necessities of the surface street under the structure, from Santa Fé Street east to the bluff, required a transverse spacing of columns which placed one column almost midway under the street-car tracks and the other under the roadway. Thus, the columns, girders, and other parts of one side of the viaduct are much larger and heavier than those of the other side.

All parts of the structure, which could be so constructed, are designed for continuous action. The upper girders were figured as continuous beams for the various combinations of span lengths, assuming a uniform moment of inertia, a condition substantially existing; and the various sections of the girders were proportioned for the figured moments. The lower-deck girders were considered as beams restrained at their ends; the end sections were proportioned for the moment of truly fixed beams, and the center sections for a moment 10% in excess of the actual figured moment for the same condition.

The cross-girders and cantilevers of the upper deck were designed for the usual continuous conditions existing, and the lower-deck cross-girders as simple beams between the supporting girders. Considering longitudinal girders alone, it is easily apparent that the great number of spans of different length, and the varying combinations and num-

bers of spans between expansion points, required rather voluminous calculations for design. It may be of interest to state that the calculations for this structure cover about 300 closely-written, 10 by 16-in. sheets.

Two features of special interest, developed in the design, are the investigation of the columns and the calculation of the arch span at Santa Fé Street.

A.—Investigation of Columns.

The great height of part of the structure and the omission of all diagonal bracing in vertical planes, either transversely or longitudinally, made necessary a thorough investigation of the stresses due not only to every combination of live loads, to wind loads, and to changes in temperature, but especially to the secondary stresses from these loads, due to the rigid connections between the columns and the upper-deck and lower-deck cross-beams and longitudinal girders. It was clearly unnecessary to go to the extreme of attempting to design every separate column by calculating its every possible stress; so that a typical series of columns fairly representing the average was selected, and calculations, as nearly complete as might be, were made for them, in order to ascertain the maximum unit stresses that could be allowed for the direct loads as a simplified basis for column design, which would give results satisfactorily accurate. The essential features of this development are here presented in abbreviated form.

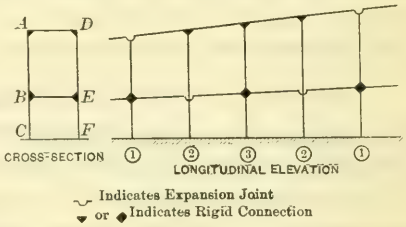


FIG. 1.

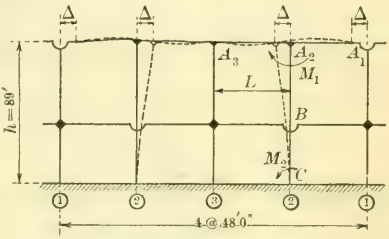
Fig. 1 shows a typical longitudinal section and cross-section of the viaduct. It will be noted that expansion joints are placed at every other column for the lower deck, thus eliminating all temperature stresses in columns, due to the shortening or lengthening of the lower deck. The upper deck has expansion joints generally at every fourth column, these joints staggering with those of the lower deck; all expansion joints for both decks are what may be called "double" expansion joints, the girders on both sides of the column being on sliding plates. The columns numbered (2) are, therefore, the only ones which have

stresses due to temperature changes. A complete investigation of this bent, therefore, was carried through.

To simplify the calculations, it will be assumed that all bents are of equal height and that the dimensions are as shown on Fig. 2.

Under the influence of a fall in temperature, the tops of Columns 2 will move toward the top of Column 3 equal distances, which are called Δ. Assuming that the lower-deck girders rest on frictionless rollers at the points, B, Columns 2 will deflect as if the lower deck were omitted.

Consider the upper girder between Columns 2 and 3; the deflection of Column 2 produces a moment, which will be called M_1, at the point, A_2. As the structure is symmetrical about the point, A_3, the tangent to the deflection curve will remain horizontal at that point, or, in other words, the girder may be considered fixed at the point, A_3; and it follows that the moment in the girder at the point, A_3, is equal to one-half the moment in the girder at the point, A_2, giving a point of contraflexure, $\frac{2}{3} L$, from A_2.



⌣ Indicates Expansion Joint
▾ or ◆ Indicates Rigid Connection

FIG. 2.

It will simplify matters to substitute for Girder A_3 A_2, a girder which is rigidly connected to the column at A_2 and is freely supported on an expansion bearing at the other end; the length of girder must be such as to give the same moment, M_1, at Column 2. This length will be $\frac{3}{4} L$. It is somewhat more simple to work with a girder length equal to L, so it will be assumed that the girder has a length, L, and that its moment of inertia, I, is increased to $\frac{4}{3} I$.

Evidently, the moments in the column will be unchanged if, for the two girders connecting to the column at A_2, one girder is substituted having a moment of inertia equal to their sum; the moment of inertia of this one girder will be taken, therefore, as $\frac{4}{3} I + I = \frac{7}{3} I$, where I is the moment of inertia of the actual girder.

As the columns are battered from top to bottom on all faces, there is a considerable variation in moment of inertia throughout their

lengths. Any formulas that are applied for the solution of the desired moments, M_1 and M_2, must take into account this variation in moment of inertia of the column. Consider the column and attached girder as a beam on three supports having moments, M_0, M_1, and M_2, at the three supports and having a straight-line moment variation, as indicated in Fig. 3; the beam is assumed to be fixed at C.

Fig. 4 shows the beam as deflected by the moments.

Bearing in mind that the deflection of any point, as C, with respect to a tangent at A_2, is equal to the moment about A_2 of the area under the $\frac{M}{E\,I}$ curve, we can write the following equations:

$$\delta_1 = \int \frac{M\,dx}{E\,I}.$$

But, $M = \frac{M_1\,x}{l}$, and I is constant for this span, therefore, we have

$$\delta_1 = \frac{M_1\,l^2}{3\,E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

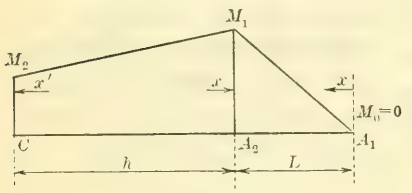


FIG. 3.

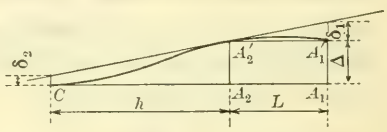


FIG. 4.

similarly, $$\delta_2 = \frac{M_2}{E}\int\frac{x'\,dx'}{I} + \frac{M_1}{E\,h}\int\frac{x'^2\,dx'}{I} - \frac{M^2}{E\,h}\int\frac{x'_1{}^2\,dx'}{I} \quad \ldots (2)$$

We have also the relation, $$\frac{\delta_1}{l} = \frac{\Delta - \delta_2}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

We can eliminate the unknown quantities, δ_1 and δ_2 from Equations (1), (2), and (3), and have the following resulting equation:

$$\frac{M_1}{E}\left(\frac{l}{3\,E\,I} + \frac{1}{h^2}\int\frac{x'^2\,dx'}{I}\right) + \frac{M^2}{E}\left(\frac{1}{h}\int\frac{x'\,dx'}{I} - \frac{1}{h^2}\int\frac{x'^2\,dx'}{I}\right) = \frac{\Delta}{h} \quad \ldots (4)$$

A second equation involving only the unknown M_1 and M_2 can be written by considering the deflection of A_2 with respect to the tangent at C, as follows:

$$\frac{M_1}{E}\left(\frac{1}{h}\int\frac{x\,dx}{I} - \frac{1}{h^2}\int\frac{x^2\,dx}{I}\right) + \frac{M_2}{E}\left(\frac{1}{h^2}\int\frac{x^2\,dx}{I}\right) = \frac{\Delta}{h} \quad \ldots\ldots (5)$$

Numerical substitution in Equations (4) and (5) results readily in a solution for values of M_1 and M_2, the desired moments.

It might be of interest to explain the method used in ascertaining the values of the integrals, $\int \frac{x\,dx}{I}$, $\int \frac{x^2\,dx}{I}$, etc.

These integrals are plotted to scale on Fig. 5. The process of plotting is as follows:

1. Plot values of I, b represents the bottom of the girder—the deformation between A_2 and b was neglected, that is, I was assumed to be infinite.

2. Plot values of $\frac{1}{I}$.

3. Plot values of $\int \frac{dx}{I}$; to locate a point of the $\int \frac{dx}{I}$ curve, measure the area between the $\frac{1}{I}$ curve, the line, AC, and the ordinate at the point; for instance, f on the $\int \frac{dx}{I}$ curve represents the area, $bcde$.

4. Plot values of $\int \frac{x\,dx}{I}$ by measuring areas between the AX axis and the $\int \frac{dx}{I}$ curve; for instance, h on the $\int \frac{x\,dx}{I}$ curve represents the area, $Abfg$.

5. The value of $\int \frac{x^2\,dx}{I}$ was obtained by measuring the whole area between the $\int \frac{x\,dx}{I}$ curve and the AX axis.

The differential areas are shown on Fig. 5 and make clear, without further explanation, the correctness of these methods.

Stresses Due to Wind Loads.—In calculating these stresses, we have a 2-story framed bent to consider; the fact that the columns have a variable moment of inertia complicates the problem, and it is necessary to use formulas taking this into account. Further complication results from the fact that the two columns in the bent are unlike, with different elastic properties.

To solve the problem and ascertain the desired moments in the columns and cross-girders, the theorem of three moments is applied.

The bent under consideration is shown in Figs. 6 and 7.

Under the influence of the wind loads, H_1 and H_2, the frame will deflect in some such manner as that shown in Fig. 7. In this diagram

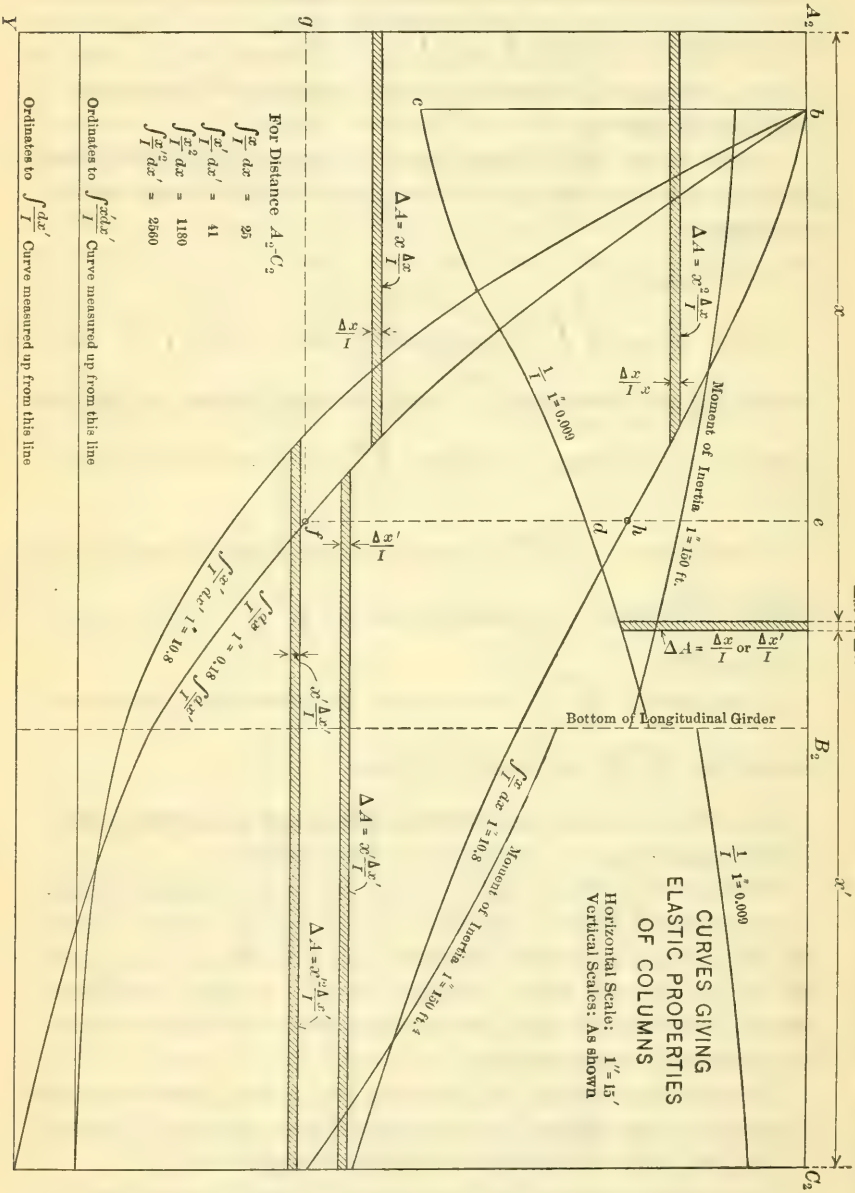

CURVES GIVING
ELASTIC PROPERTIES
OF COLUMNS
Horizontal Scale: 1" = 15'
Vertical Scales: As shown
Moment of Inertia I" = 150 ft.
Bottom of Longitudinal Girder
For Distance A_2-C_2
$\int \frac{x}{I} dx$ = 25
$\int \frac{x'}{I} dx'$ = 41
$\int \int \frac{x'}{I} dx'$ = 1180
$\int \int \frac{x''}{I} dx'$ = 2560
Ordinates to $\int \frac{x dx'}{I}$ Curve measured up from this line
Ordinates to $\int \frac{dx'}{I}$ Curve measured up from this line
Δx or $\Delta x'$

FIG. 5.

also are shown the unknown moments, M_0 to M_7, for which we must solve.

The 2-story frame, shown in Fig. 7, can be broken up into two separate frames, *ABED* and *BCFE*; in the consideration of the upper frame, *ABED*, M_2 and M_3 must be considered as external moments acting at *B* and *E*; similarly, for the lower frame, M_5 and M_4 are the external moments. Using the formulas for continuous beams, we can now write four equations for each frame, or a total of eight equations. We have ten unknowns in these eight equations, the moments, M_0 to M_7, and the deflections, δ_1 and δ_2. The two additional equations neces-

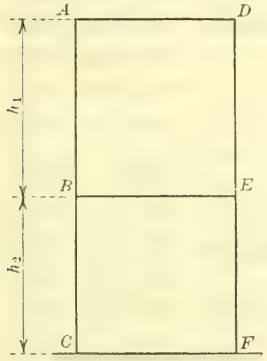


FIG. 6.

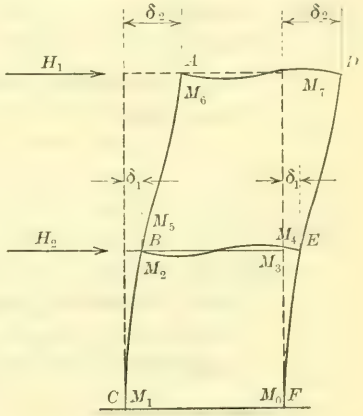


FIG. 7.

sary to solve can be obtained from the static equilibrium conditions, as follows:

$$H_1 h_1 = M_4 + M_5 + M_6 + M_7$$
$$(H_1 + H_2)h_2 = M_0 + M_1 + M_2 + M_3.$$

By arranging these ten equations in tabular form, the solution is found with comparatively little labor.

Unit Stresses in Columns.—The methods indicated for the calculation of temperature and wind stresses were in general used to evaluate the column bending stresses due to dead and live loads on the main girders and cross-girders. The columns are battered $\frac{1}{8}$ in. per ft. on all four faces, and pilasters are added below the lower-deck girders; the result is that the maximum unit stresses in the columns occur at their tops, on a line with the bottom of the longitudinal girders. Various probable combinations of loading were considered; the maxi-

mum probable stresses that would ever occur in the columns were assumed to be covered by the following two cases:

Case A.—Live load in position to produce maximum reaction on column; transverse wind at 15 lb. per sq. ft.; temperature change of 40° Fahr.

Case B.—Live load on upper deck in position to produce maximum bending in a longitudinal direction; live load on lower deck in position to produce maximum reaction on column; transverse wind at 15 lb. per sq. ft.; temperature change of 40° Fahr.

The stresses for these two cases for a typical column were:

Case A:	Direct	355 *C*		355 *C*
	Wind	136 *C*	or	136 *T*
	Rigid cross-girder connection.	86 *C*	or	86 *T*
	Temperature	97 *C*	or	97 *T*
		674 *C*	or	36 *C*
Case B:	Direct	300 *C*		300 *C*
	Wind	116 *C*	or	116 *T*
	Rigid cross-girder connection.	86 *C*	or	86 *T*
	Temperature	97 *C*	or	97 *T*
	Longitudinal girder connection.	182 *C*	or	182 *T*
		781 *C*	or	181 *T*

These stresses were figured on the basis of a plain concrete column, the reinforcement being neglected, and, considering the extreme conditions of loading, were thought to be satisfactory.

As a result of these investigations, the sections of the columns were determined on the following basis:

The column section under the cap was determined by providing for the maximum direct load on the column at 350 lb. per sq. in., steel reinforcement not being considered. The section above the column cap was made not less than 92% of the section under the cap. The columns were reinforced with $1\frac{1}{4}$-in. round bars, spaced at approximately equal distance around the perimeter of the column, so as to give a percentage of 0.8 of 1% at the point of minimum section on the heavy column of any bent. The steel for the light columns was then determined by using the same spacing as for the heavy column.

B.—Design of Arch.

Other designing problems of special interest are in connection with the arch at Santa Fé Street. This span has a clear opening of 130 ft., and consists of two arch ribs transversely spaced so that the lower roadway passes between them.

The two arch ribs support spandrel columns which, in turn, carry cross-girders of the usual type and the cantilevers of the upper deck. The lower deck is suspended from the arch ribs. Because of the very limited distance between the surface of the roadway on the lower deck and the clearance required above the railroad tracks below, the lower deck is composed of steel I-beams encased in, and carrying a slab of, concrete. These cross-beams rivet into continuous longitudinal steel beams which are supported at intervals of 17 ft. by hangers composed of two steel channels.

The horizontal thrusts of the arches are taken by the bottom chords of the steel eye-bars. This avoids the use of abutments large enough to resist the thrust—there being in fact insufficient room for such abutments—and affords several advantages because of the resulting ability to shorten the lower chords. Because of the short length of the hangers near the springing points of the arch, and the rigidity of the lower-deck girder, it was considered advisable to make the arches of the fixed type. One end of each arch rib is fixed by being built into the supporting pier, and the other end rests on a tandem rocker bent placed so as to keep the resultant thrust of the arch between the two bearings for every condition of loading. The springing points of the arch are relatively on the same grade as the lower roadway, so that the east springing point is 3.5 ft. higher than the west one. This condition causes some unusual longitudinal thrusts.

All the structural steel members of the span are encased in concrete, and the smaller ones are surrounded by light concrete walls giving dimensions for satisfactory architectural appearance.

The arches were designed under the usual elastic theory. The calculations were carried through for both unit vertical and unit horizontal loads at the various points, vertical loads in this instance meaning those perpendicular to, and horizontal loads meaning those parallel to, the arch axis. These latter loads include those due to thrust from the street-car loading, as well as the components of the dead and live loads caused by the grade.

The maximum stresses were calculated at each springing point, at the crown, and at seven intermediate points on each half of the ring—a total of seventeen different points. All calculations were carried through in appropriate tabular form, and are hardly of sufficient interest to warrant their inclusion in this paper. The influence lines for maximum moments at various points, due both to vertical and to horizontal loads, were plotted and are shown on Plate XI.

Considerable experience in actual designs recommends highly the influence-line methods for a check on calculations. Errors seem to be invariably made evident by the failure of the various ordinates to any influence line to show a smooth curve. In the diagrams, it will also be noted that the apices of the various moment curves themselves lie on a smooth curve, a further check on the numerical work.

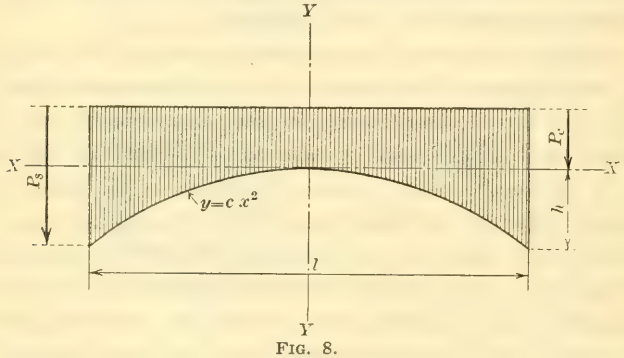


FIG. 8.

It was assumed that the intensity of loading on the arch ring, including the weight of the ring itself, would vary as the ordinates of a parabola, a condition of loading shown by Fig. 8; and the center line of the arch ring was located in the following manner:

Let P_c = equivalent uniform load at the crown;

P_s = equivalent uniform load at the springing;

$$u = \frac{P_s}{P_c};$$

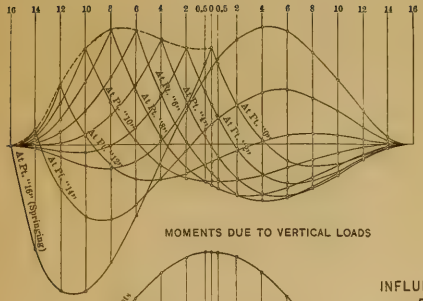
h = rise of arch ring = 24 ft.;

l = span of arch ring = 134 ft.

Then, the equation of the funicular polygon results as follows:

$$y = \frac{4\,h}{l^2\,(u+5)}\left[6\,x^2 + (u-1)\,\frac{4\,x^4}{l^2}\right].$$

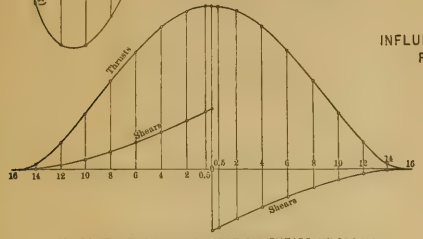
This equation was used as the equation for the center line of the arch ring, using dead load plus one-half live load in substituting



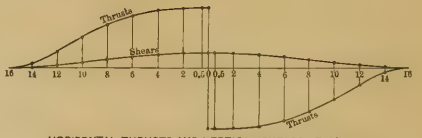
MOMENTS DUE TO VERTICAL LOADS

MOMENTS DUE TO HORIZONTAL LOADS

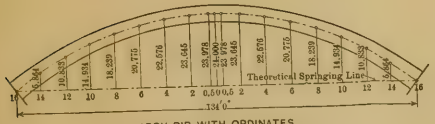
INFLUENCE LINE DIAGRAMS FOR ARCH SPAN



HORIZONTAL THRUSTS AND VERTICAL SHEARS AT CROWN DUE TO VERTICAL LOADS



HORIZONTAL THRUSTS AND VERTICAL SHEARS AT CROWN DUE TO HORIZONTAL LOADS



ARCH RIB WITH ORDINATES

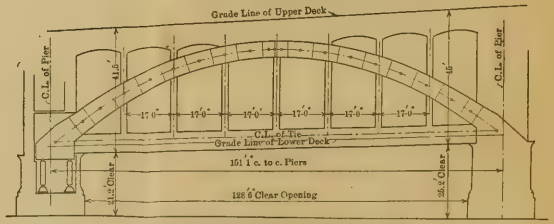


DIAGRAM OF GENERAL DIMENSIONS OF ARCH SHOWING ASSUMED DIVISIONS OF ARCH RING

the numerical values of P_c and P_s. This substitution gave slightly different values of u for the two arch rings; and therefore an average value of $u = 1.34$ was used.

The equation was further simplified for use by dividing each half of the arch ring into sixteen equal horizontal sections, and numbering the division points successively from the crown to the springing.

Let N = number of any division point;
= 0 at crown;
= 16 at springing;

Then, $X = \frac{N\,l}{32}$.

Substituting the values of u, h, l, and x in the original equation, we have the following equation for the center line of the arch ring:

$$y = 0.0887\ N^2 + 0.196\ \frac{N^4}{10^4}.$$

Similarly, the general equation for the tangent at any point of the arch ring, which is,

$$\frac{dy}{dx} = \text{tan. } \phi = \frac{16\ h}{l^2\ (u + 5)} \left[3\ x + (u - 1)\ \frac{4\ x^3}{l^2} \right]$$

simplifies to,

$$\text{tan. } \phi = 0.0423\ N + 0.01875\ \frac{N^3}{10^3}.$$

The lower deck of the arch span is on a grade of 2.61%, and in locating the center line of the arch ring, the X axis was made parallel to the grade and the Y axis perpendicular to it. As the hangers supporting the lower deck are vertical, it is evident that the dead loads are not applied symmetrically with respect to the crown of the arch ring. For purposes of calculation, it was assumed that the loads were applied symmetrically; this resulted in an error of less than 1%, which was considered negligible.

The use of the bottom chords of the eye-bars to take the thrust eliminates all temperature stresses in the arch rings, with the exception of those due to a possible variation in temperature between the arch ring and the bottom chord, which were considered negligible.

Toggle arrangements to shorten the lower chords were provided to accomplish the following:

1.—To take up the stretch in the bottom-chord eye-bars, due to dead load, the result being to leave the dead-load stresses in the arches the same as for fixed arches supported on rigid abutments.

2.—To shorten the bottom chords by amounts sufficient to eliminate the stresses in the arch rings due to their shortening under dead load.

3.—To shorten the bottom chords by the amounts which they would stretch, and the arch ribs would shorten, under full live load, over one-half the span.

As indicated by the influence lines for maximum moments at various points of the arch ring, the loaded length for a maximum moment is generally about one-half the span length; the result of shortening the bottom chord as indicated under Paragraph 3 is, therefore, to give practically the same maximum stresses in the arch ring as would result in a fixed arch supported on abutments in the ordinary way, with the further advantage that the stresses due to rib shortening are eliminated.

All these provisions required a total shortening of the bottom chords by the toggles of $1\frac{1}{16}$ in., which meant that the distance between the springing points, after the dead load was placed on the arches, was $\frac{3}{8}$ in. shorter than when the arches were constructed on the falsework.

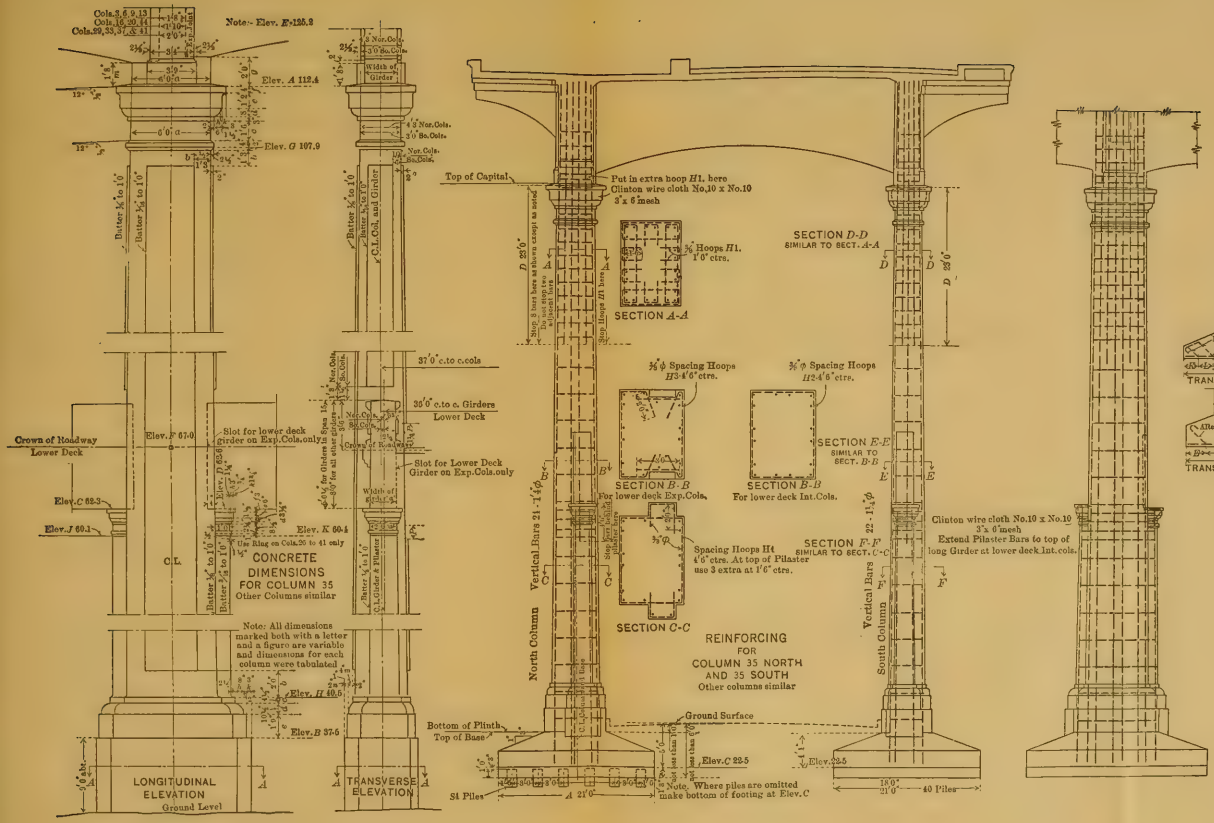
4.—Specifications for Materials and Workmanship.

The specifications were prepared with a view to letting a unit-price contract after competitive bidding. Every effort was made to have them full and precise in description, both of materials and of workmanship, eliminating as far as possible the uncertain characterization of work "satisfactory to the Engineer" by expressing, wherever feasible, the views of the engineers in advance, and attempting to make perfectly clear to a prospective builder just what would be required of him. Though definite and explicit in the description of results desired, the specifications left open as far as possible the methods by which such results are to be secured, in order that the City might receive the full benefit of any contractor's skill and ingenuity in laying out and handling the work, and enjoy a portion of the economies thus effected.

Approximate quantities of various units of the structure were prepared and tabulated by the engineers. The City received bids on a unit basis of price per cubic yard for concrete in various divisions of the structure, price per pound for reinforcing material, for structural steel, etc., and the total bids were compared on the basis of the approximate quantities.

PLATE XII.
PAPERS, AM. SOC. C. E.
MAY, 1915.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.

TYPICAL DETAILS OF COLUMNS, CAPS, AND BASES TWELFTH STREET TRAFFICWAY VIADUCT KANSAS CITY, MO.


Note:- Elev. E-125.2
Elev. A 112.4
Elev. G 107.9
Batter ¾" to 1'0"
C.L. Col. and Girder
Top of Capital
Put in extra hoop H1, here
Clinton wire cloth No.10 x No.10
3" x 6" mesh
Hoops H1, 1'0" ctrs.
SECTION A-A
SECTION D-D
SIMILAR TO SECT. A-A
37'0" c. to c. cols
36'0" c. to c. Girders
Lower Deck
Crown of Roadway
Lower Deck
Elev. F 67.0
Slot for lower deck Girder on Exp. Cols. only
Slot for Lower Deck Girder on Exp. Cols. only
Elev. C 62.3
Elev. J 60.1
Elev. K 60.4
C.L.
CONCRETE DIMENSIONS FOR COLUMN 35
Other Columns similar
Note: All dimensions marked both with a letter and a figure are variable and dimensions for each column were tabulated.
Elev. H 40.6
Elev. B 37.0
LONGITUDINAL ELEVATION
Ground Level
TRANSVERSE ELEVATION
¾" ϕ Spacing Hoops H3 4'6" ctrs.
¾" ϕ Spacing Hoops H2 4'6" ctrs.
SECTION B-B
For lower deck Exp. Cols.
SECTION B-B
For lower deck Int. Cols.
SECTION E-E
SIMILAR TO SECT. B-B
Spacing Hoops H4 4'6" ctrs. At top of Pilaster use 3 extra at 1'6" ctrs.
SECTION C-C
SECTION F-F
SIMILAR TO SECT. C-C
REINFORCING FOR COLUMN 35 NORTH AND 35 SOUTH
Other columns similar
North Column
Vertical Bars 21-1¼ϕ
Vertical Bars 22-1¼ϕ
South Column
Clinton wire cloth No.10 x No.10
3" x 6" mesh
Extend Pilaster Bars to top of long Girder at lower deck Int. cols.
Bottom of Plinth
Top of Base
Ground Surface
Elev. C 22.5
Elev. 22.5
Note. Where piles are omitted make bottom of footing at Elev. C
84 Piles
A 21'0"
18'0"
21'0"
40 Piles

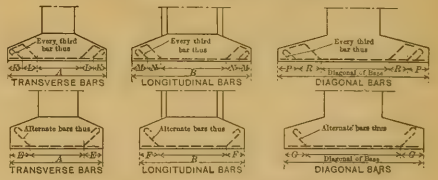
LOADS ON FOUNDATIONS

BASES WITH PILES

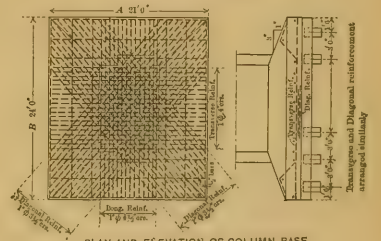
Average load on piles in each base, 25 tons per pile
Maximum load, considering effect of wind, temperature changes, and deflection of cross-girders, 35 tons per pile

BASES WITHOUT PILES:

Pier No.	Loads in Pounds per Square Foot	
	Av.	Max.
39, 40, 41, 43, 45	12 000	14 000
42	8 000	11 000
44	6 500	9 000



BENDING DIAGRAMS FOR BASE REINFORCEMENT



PLAN AND ELEVATION OF COLUMN BASE

Concrete.—The specifications required the concrete of the structure to be made of 1 part cement, of detailed requirements, conforming in most respects to the recommendations of the American Society for Testing Materials; of 2 parts sand, "particles of hard, clean stone in graded sizes which shall pass a sieve having holes $\frac{1}{4}$ in. square, and not less than 50% be retained on a No. 30 sieve"; and 4 parts broken stone, "pieces of hard and durable rock entirely free from clay, loam, or any other material than sound, hard stone—of crusher-run sizes—with all material that will pass a $\frac{1}{4}$-in. sieve removed", to pass a $1\frac{1}{4}$-in. ring.

For the hand-rails, stair risers, and treads, a proportion of 1 part cement, $1\frac{1}{2}$ parts sand, and 3 parts broken stone, to pass a $\frac{3}{4}$-in. ring, was required. The proportions are stated to be by volume, with the ingredients measured loose, one 380-lb. bbl. of cement being taken as 4 cu. ft. The upper $\frac{3}{4}$ in. of the sidewalk slab is of mortar of 1 part cement to $1\frac{1}{2}$ parts sand placed before the supporting slab had set. The use of a mechanical batch mixer is specified, to be operated long enough after the last ingredient is deposited in it thoroughly to mix and incorporate all ingredients.

Reinforcement.—The reinforcement is of corrugated bars, "deformed bars having their corrugations at right angles to the length of the bar" having been required, rolled from open-hearth billets of medium steel having an ultimate strength of from 60 000 to 75 000 lb., and a minimum elastic limit of 35 000 lb., and affording an elongation of not less than 22% in 8 in.

The specifications provide that the "reinforcement in the finished structure shall conform accurately in size and position to the requirements of the plans. Bars shall be firmly bound and tied together by wire where they lap or cross, or be fastened by clips or other devices. Each piece must be held rigidly and positively in position, so that there will be no displacement during the depositing of concrete".

Concrete Piles.—Precast reinforced concrete piles were designed, and were detailed on the drawings, but the specifications left some option to the contractor as to the exact type to be used, excluding, however, "piles in which fresh or unset concrete is placed against the soil". The contractor finally elected to use Raymond piles which were accepted by the engineers.

A description of this well-known concrete pile is hardly necessary, but it may be said briefly to consist of a sheet-steel shell, spirally

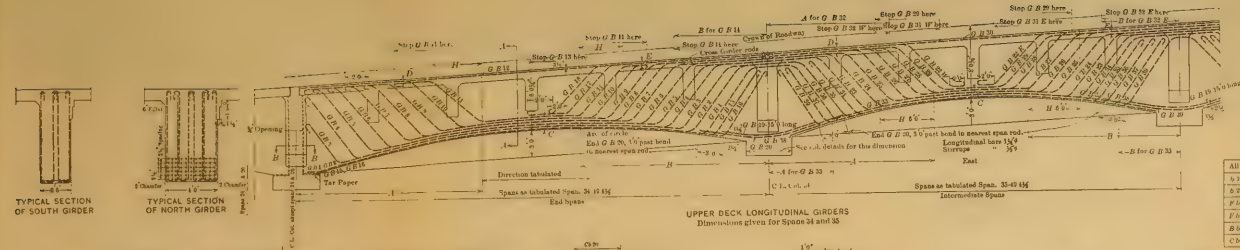
reinforced, driven into the ground on a collapsible steel core. After reaching a satisfactorily resisting penetration, the core is withdrawn and the shell is filled with concrete. No reinforcing rods in the concrete were required, and as the concrete was not to be disturbed after placing, a mixture 1 part cement, 3 parts sand, and 5 parts broken stone was permitted.

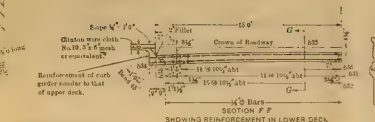
Pavement.—The specifications for the creosoted wood block pavement for the lower roadway and on the street contain no unusual features. For steeper grades the following requirements are of interest:

"Creosoted block pavements, for roadways having a gradient exceeding 4%, are to be constructed in all parts of material and as specified for level roadways with the following exceptions: Creosoted laths about 1¼ in. wide, ⅜ in. thick, and 4 ft. long, surfaced on one side, of uniform thickness, are to be furnished and placed on edge between every row of blocks continuously across the roadway, leaving a space about ⅜ in. wide and 2¾ in. deep between every row of blocks. The spaces between the blocks shall be filled with hot gravel which has passed through a sieve having holes ¼ in. square, and from which all sand and fine material which will pass a No. 20 sieve has been removed." * * * "Immediately after the gravel is in place, before it has cooled, paving pitch, as herein specified, shall be poured into it in such manner and quantity as entirely to fill all interstices."

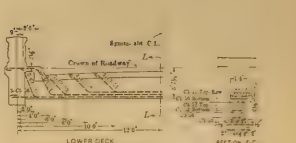
This is practically the form of block pavement used in Vancouver, B. C., and elsewhere, on heavy grades. It affords a good grip for horses, and has proved very satisfactory.

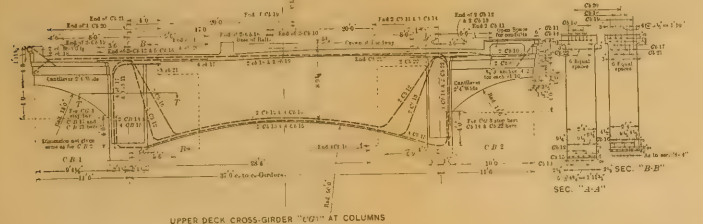
Water-proofing.—The surfaces of both decks are covered with water-proofing in an effort to prevent the dripping of water on the roadways below. Under the ballast of the street-car area, this water-proofing consists of a two-ply burlap mat, laid down in water-proofing pitch, on which is placed a layer of asphaltic felt, and this, in turn, is covered with 1 in. of asphaltic mastic, and given a sand finish. Under the creosoted block pavement, one layer of burlap and one layer of asphaltic felt are laid in water-proofing pitch and given a sand finish. Indentations are formed in the concrete curbs some 6 in. below their tops, and the water-proofing mats are brought up, turned in, and finished in them. It happened that just before the contractor was to place his order for the burlap, the supply was entirely cut off owing to the European war conditions, so that felt was substituted, at some reduction of cost.

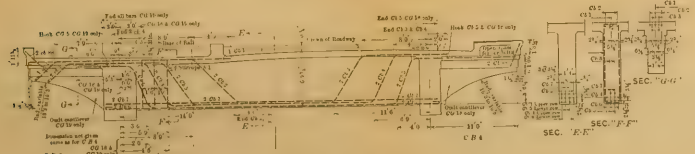


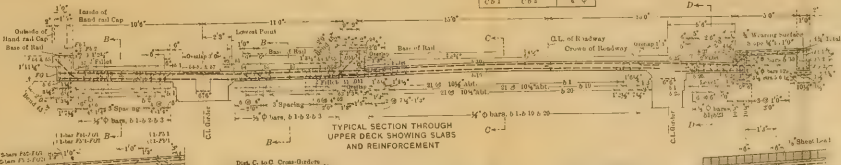

Slope ¼" / 0"
Fillet
Crown of Roadway
Glinton wire cloth No. 19 3 x 6 mesh or equivalent
Reinforcement of curb girder similar to that of upper deck.
½" Bars
SECTION E F
SHOWING REINFORCEMENT IN LOWER DECK
TYPICAL DETAILS OF
LOWER DECK FLOOR-SLABS

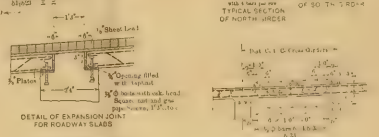
All bars ½" ⌀ except as follows	
b 19	b 16
b 29	b 30
F 0-1	F 0-2
F 0'1	F 0'2
B 0-1	B 0-2
C b 1	C b 2

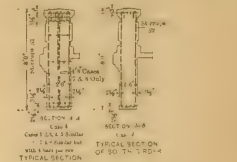


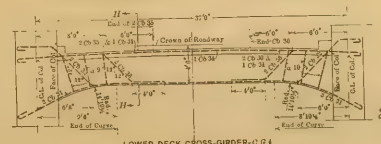


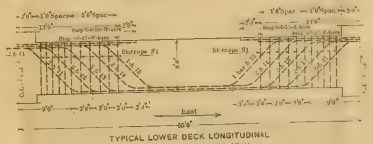












TYPICAL DETAILS OF SLABS GIRDERS AND LONGITUDINAL GIRDERS FOR BOTH DECKS TWELFTH STREET TRAFFICWAY VIADUCT. KANSAS CITY, MO.

Lighting System.—The requirements for the lighting system were given as follows:

"The structure is to be illuminated by an electric lighting system. There shall be furnished and installed, all lamps, globes, conduits, wiring, and all other apparatus and appurtenances necessary for a complete series-Tungsten lighting system, taking current from the Kansas City Electric Light Company's 6.6-ampere, constant-current feeders at the east end of the structure. There will be two circuits. The circuit for lighting the upper roadway will have sixty 125-watt, 6.6-ampere, constant-current series-Tungsten lamps. Each lamp will be supported on a cast-iron standard and will be surrounded by an opal glass globe 14 in. in diameter. The circuit for lighting the lower roadway and the stairways will have fifty-six 75-watt, 6.6-ampere, constant-current, series-Tungsten lamps. Each lamp will be supported on a cast-iron standard or bracket and will be surrounded by an opal glass globe, 12 in. in diameter. All wiring shall be drawn into Sherardized steel-pipe conduits of a satisfactory type, so as to be free from all flaws or mechanical injuries. All wires shall be No. 6, high-tension, lead-covered, Okonite stranded copper wire of the best quality. The conduit shall be encased in the concrete as the latter is placed, and shall be properly supported and arranged so that, later, the wire may be drawn into place. Suitable expansion joints shall be provided, and all connections shall be made in the conduits so that they will conform properly to the requirements of the structure. All joints between pipes and between pipes and boxes shall be made water-tight. The entire lighting system shall be constructed in a thoroughly workmanlike manner and to the satisfaction of the engineers."

Other features of the specifications will be evident from the descriptions of methods of construction.

5.—Equipment for, and Methods of, Construction.

Yard Space.—The contractor's plant was at the west or low end of the viaduct, where some storage space, switch tracks, sidings, and facilities for handling materials, were more easily available than elsewhere on the right of way. The working space comprised practically all the ground from Liberty Street to Hickory Street, and from the north curb line of Eleventh Street to the alley south of Twelfth Street, an area of about 35 000 sq. ft.

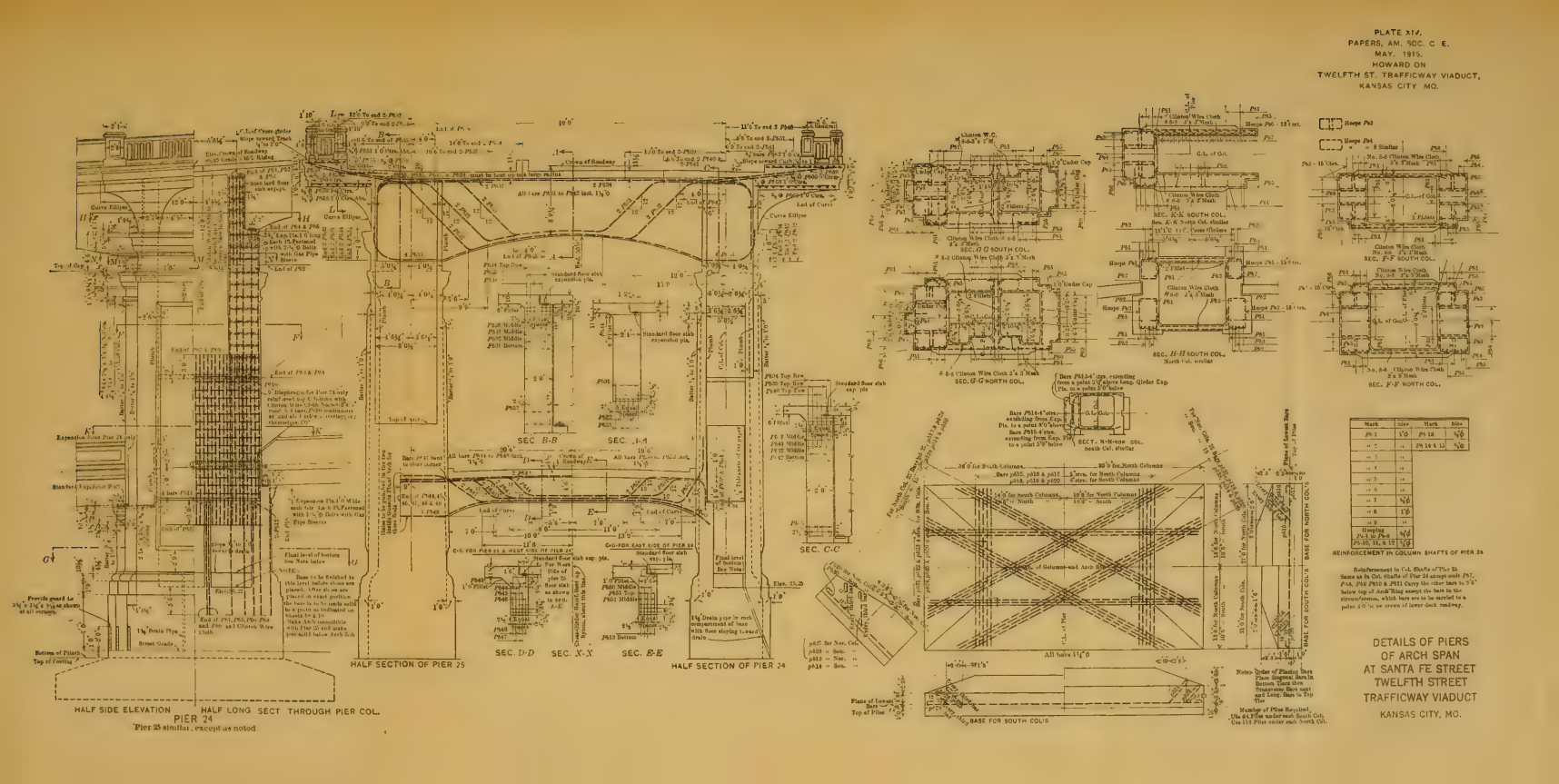
Three standard-gauge spur tracks were provided, one track paralleling Twelfth Street in the alley south of it—used for the delivery of timber; and two tracks crossing Twelfth Street at right angles—one for the delivery of sand, stone, and steel, and the other for cement.

Concrete Plant.—The mixing plant, placed on the line of the viaduct, consisted of elevated bins for sand and broken stone arranged to discharge into measuring hoppers, below which a 33-cu. ft. Chicago-cube mixer was installed. The mixer dumped the concrete into hopper cars operating on a 30-in. gauge track passing underneath. Stone and sand were delivered in bottom-dump cars and unloaded into a receiving pit alongside the bunkers. A 22-in. Jeffery, belt-bucket elevator, of a total height of 70 ft., raised the stone and the sand from the pit to the top of the bins, where a baffle-board directed each to a proper bin. One elevator thus handled both sand and stone, unloading first one and then the other. The cement was hauled up an inclined track from the cement storage shed on a car, self-dumping at the top, to a platform under the sand bin, where it was filled into a sheet-iron tube with a bottom gate, and so discharged into the mixer. The tube was of just the proper dimensions to hold six sacks of cement, the normal charge. The limited room for tracks made necessary considerable shifting of the cars, as only the first car of a train could be placed over the pit. The operations of unloading and mixing were not always simultaneous, but when all operations were going forward, there were required, severally, two men for unloading and handling the elevator, three men for unloading cement from cars, one in the storage shed, two opening sacks and feeding to the mixer, and one man attending to charging and operating the mixer. Thus, the services of nine men were needed for the unloading and mixing of concrete ingredients.

During severe weather, the stone, sand, and water for the concrete were heated by passing steam from a 60-h.p. boiler into the bottoms of the sand and stone bins, and into the water, thoroughly and satisfactorily heating all the materials.

Equipment for Steel and Timber Reinforcing.—The steel-storage yard was adjacent to one of the switch tracks, and was equipped with a working platform and a power bar-bender of ingenious design. This bender was devised on the job, and included a motor connected by a worm gear to a vertical shaft which had at its upper end a clutch with a jaw to engage the bars to be bent, thus bending the bars in a horizontal plane on a protractor plate to any desired angle. The lumber-storage yard was beside the third siding. From this lumber yard, the material was passed into a wood-saw shop, thence into the carpenter shops, and then forward to the structure. The wood shop was equipped

PLATE XIV.
PAPERS, AM. SOC. C. E.
MAY, 1915.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.



Reinforcement in Col. Shafts of Pier 25
Same as in Col. Shafts of Pier 24 except omit P87, P88, P89 P90 & P91 Carry the other bars to 3'6" below top of Arch Ring except the bars in the circumference, which bars are to be carried to a point 3'0" be ow crown of lower deck roadway.

DETAILS OF PIERS OF ARCH SPAN AT SANTA FE STREET TWELFTH STREET TRAFFICWAY VIADUCT

KANSAS CITY, MO.

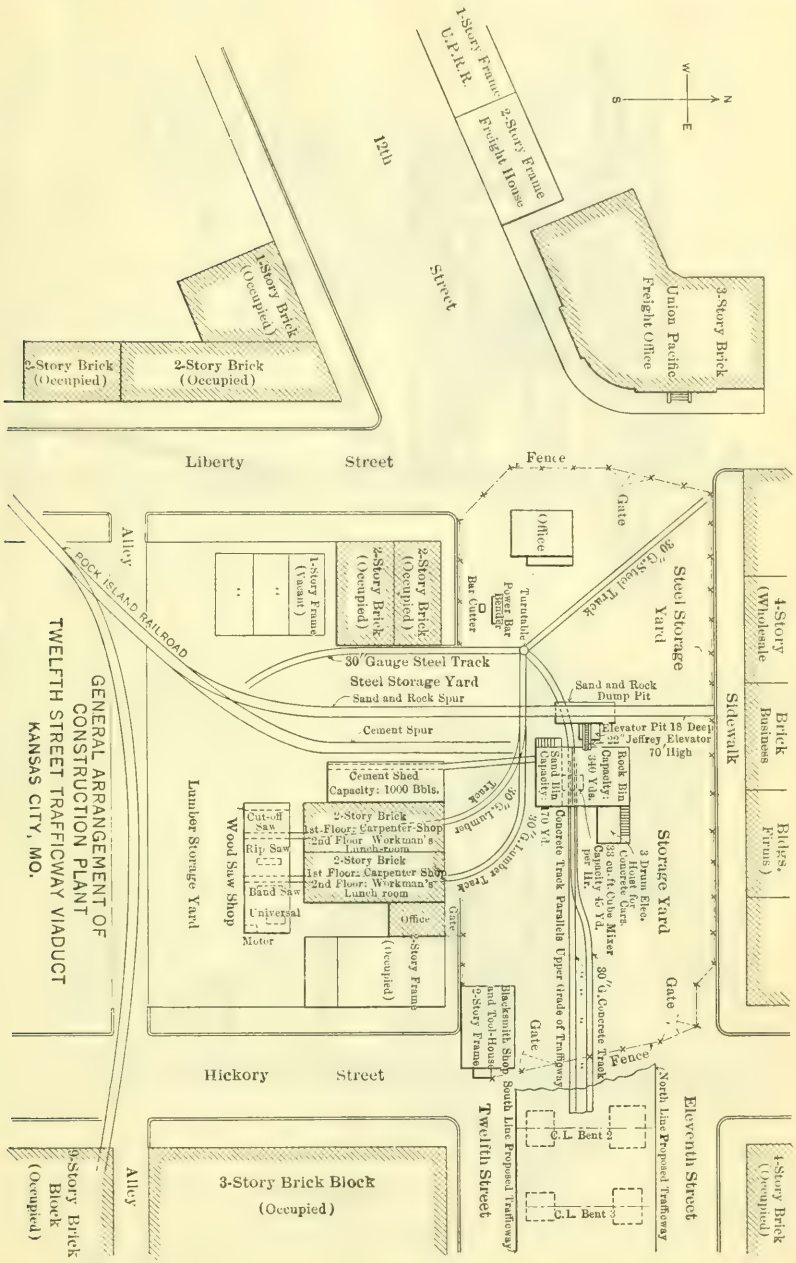

GENERAL ARRANGEMENT OF CONSTRUCTION PLANT
TWELFTH STREET TRAFFICWAY VIADUCT
KANSAS CITY, MO.
Liberty Street
Hickory Street
12th Street
Twelfth Street
Eleventh Street
Alley
ROCK ISLAND RAILROAD
1-Story Frame U.P.R.R.
2-Story Frame Freight House
Union Pacific Freight Office
3-Story Brick
1-Story Brick (Occupied)
2-Story Brick (Occupied)
2-Story Brick (Occupied)
1-Story Frame (Vacant)
2-Story Brick (Occupied)
2-Story Brick (Occupied)
Office
Fence
Gate
30' Ga. Steel Track
Steel Storage Yard
4-Story (Wholesale)
Brick Business
Bldgs. Firms
Sidewalk
Turntable
Power Bar Bender
Bar Cutter
30' Gauge Steel Track
Steel Storage Yard
Sand and Rock Spur
Sand and Rock Dump Pit
Cement Spur
Elevator Pit 18' Deep
22' Jeffrey Elevator 70' High
Rock Bin Capacity: 340 Yds.
Sand Bin Capacity 70 Yd.
Cement Shed Capacity: 1000 Bbls.
30' Ga. Lumber Track
30' Ga. Lumber Track
Lumber Storage Yard
Wood Saw Shop
Cut-off Saw
Rip Saw
Band Saw
Universal
Motor
2-Story Brick 1st Floor: Carpenter Shop 2nd Floor: Workman's Lunchroom
2-Story Brick 1st Floor: Carpenter Shop 2nd Floor: Workman's Lunch room
Office
2-Story Frame (Occupied)
Concrete Track Parallels Upper Grade of Trafficway
3 Drum Elec. Hoist for Concrete Cars
Storage Yard
35 cu. ft. Cube Mixer Capacity 40 Yd. per Hr.
30' Ga. Concrete Track
Gate
Gate
Fence
Blacksmith Shop and Tool-House 2-Story Frame
South Line Proposed Trafficway
North Line Proposed Trafficway
C.L. Bent
C.L. Bent
3-Story Brick Block (Occupied)
9-Story Brick Block (Occupied)
4-Story Brick (Occupied)

Fig. 9.

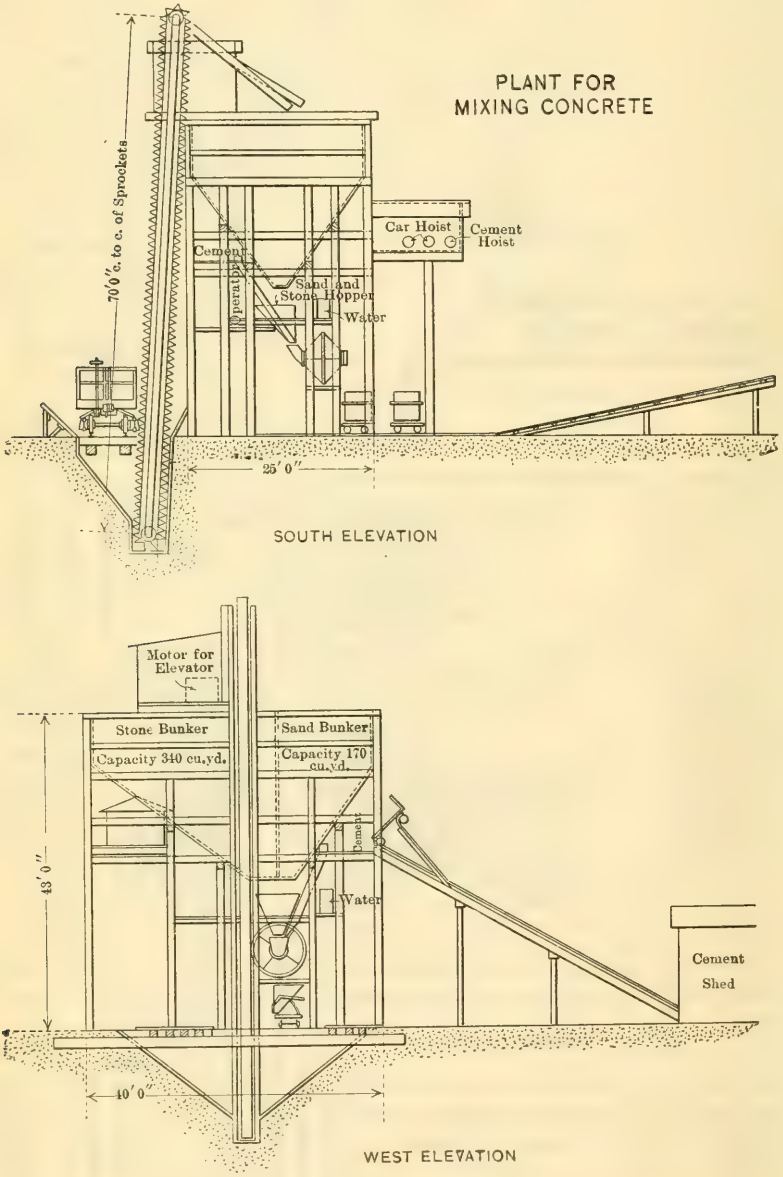
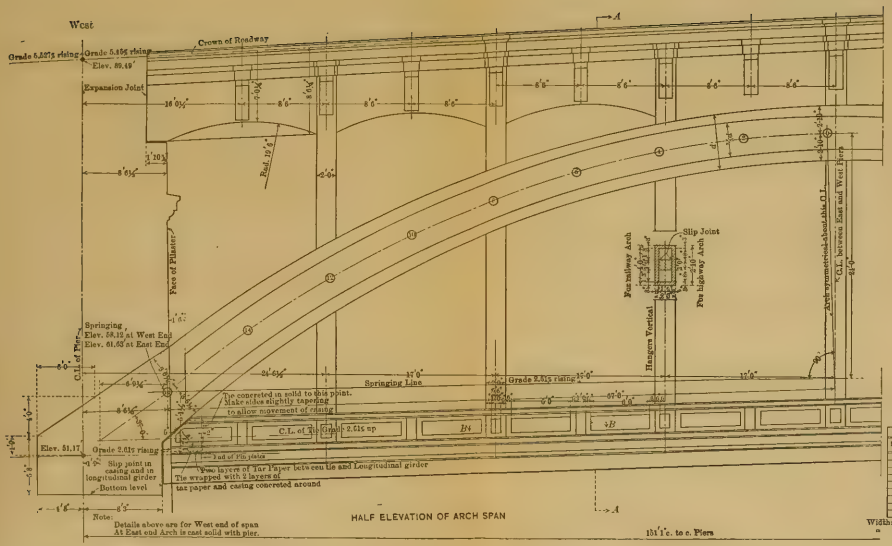

PLANT FOR
MIXING CONCRETE
70′0″ c. to c. of Sprockets
Car Hoist
Cement Hoist
Cement
Operating
Sand and Stone Hopper
Water
25′0″
SOUTH ELEVATION
Motor for Elevator
Stone Bunker
Capacity 340 cu.yd.
Sand Bunker
Capacity 170 cu.yd.
Cement
Water
43′0″
40′0″
Cement Shed
WEST ELEVATION

FIG. 10.



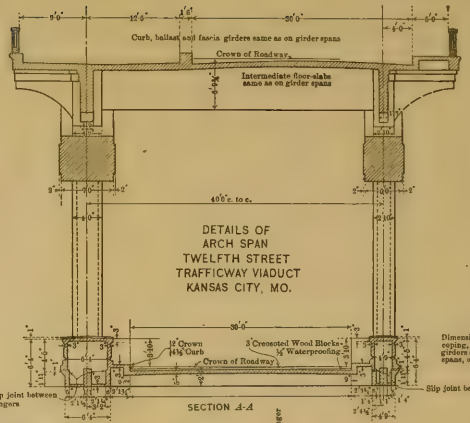


TABLE FOR ARCH RIB			
Point	Coördinates		Depth in ft.
	X in ft.	Y in ft.	
0	0	0	5.81
2	6.33	0.38	5.85
4	16.76	1.47	5.79
6	25.14	3.01	5.67
8	33.07	5.16	6.05
10	41.90	9.07	6.75
12	50.24	13.57	6.45
14	57.95	18.14	6.77
16	67.06	24.06	7.07

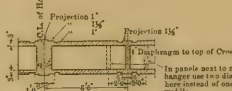
Width: Highway Arch, 5' 0" throughout
" Railway " 7' 0" "

Note.-

Abscissæ measured from crown, in both directions on line parallel to lower deck grade line.

Ordinates measured downward, perpendicular to abscissæ.

The depth is measured in radial direction.



SEC. B-B

In panels next to middle hanger use two diaphragms here instead of one in the middle.

This detail typical for entire arch, except as noted.

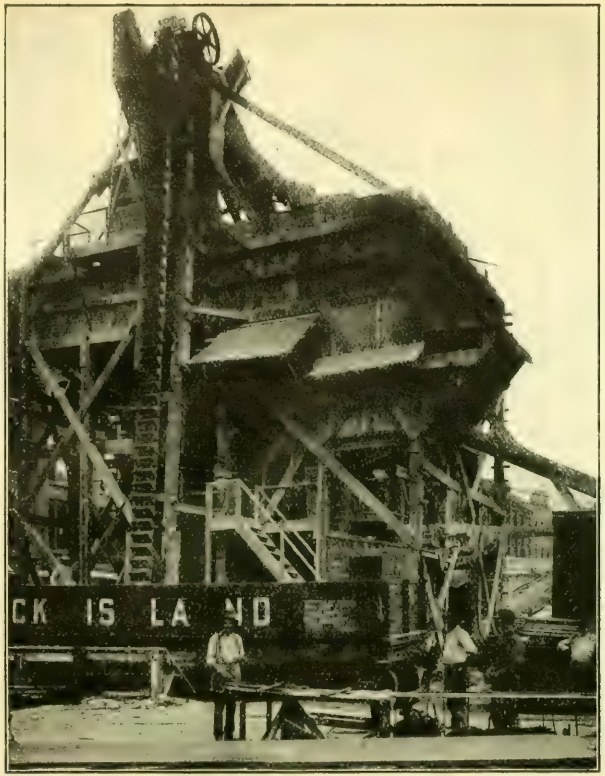


FIG. 11.—BACK VIEW OF MATERIAL BINS AND MIXING PLANT, SHOWING PIT AND BELT BUCKET ELEVATOR.

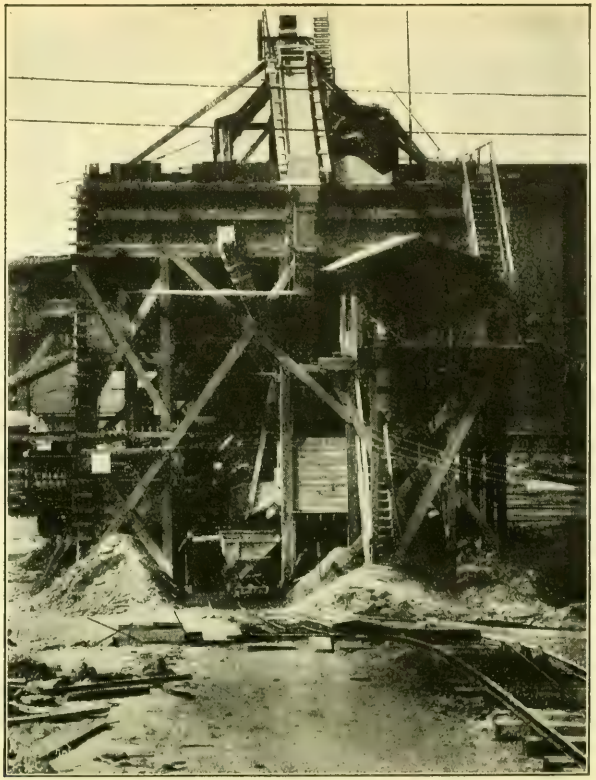


FIG. 12.—FRONT VIEW OF MATERIAL BINS AND MIXING PLANT, SHOWING MIXER, CONCRETE CAR, HOIST, ETC.

with a circular rip-saw, a "Hall and Brown rip-saw No. 131", a "Crescent" 36-in. band saw, a "Crescent" swing cut-off with 18-in. saw, and a universal edger and planer known as "Hall and Brown New Model Single Sticker No. 30".

There was also a well-equipped blacksmith shop.

Transportation.—All materials were put in form as nearly as possible ready for erection in this yard. All were conveyed out to place on a double narrow-gauge, industrial track carried on a tramway trestle some 8 ft. above the finished roadway surface, on the falsework, or on the partly finished structure. As construction proceeded, the tram tracks were placed on the upper roadway. The cars operating on this track were hauled forward and backward in pairs by a cable operating on two drums of a hoist placed on the mixing plant; the empty car being returned as the loaded car was hauled out.

For conveying concrete, four 2-yd., side, bottom-dump cars were used; and about six sets of trucks provided cars of various length for handling forms, steel, etc. Two light traveling derricks of about 2 tons capacity, one on the lower deck and the other on the upper tramway on top of the upper deck, were each equipped with electric hoists.

The entire equipment, including elevators, concrete mixer, woodworking machinery, blacksmiths' tools, bar bender, etc., was operated by electricity, alternating-current, 220-volt, 3-phase, 25-cycle being available, and individual motors being attached to the various units.

Pile-Driving Equipment.—The piles were driven with a steel-frame, turn-table driver with 56-ft. steel leads, equipped with a 42-h.p. hoisting engine and a No. 1 Vulcan steam hammer weighing 5 tons, resting on rollers supported on wooden sills laid on the ground. Timber cribbing was used to support the sills across the excavations. There were, of course, the usual collapsible pile cores and other special equipment required in the construction of Raymond piles.

With the exception of the pile-driving outfit, practically the entire equipment was purchased new for this work and was carefully selected, in order to fit exactly the service needed. The orderly progress of the work is a testimonial of the adequacy and suitability of the plant.

Procedure of Sub-Surface Work.—The excavations, for the most part, were very simple, not very deep, on account of the concrete piles, and in a sandy loam, easily dug, and standing without much shoring

or sheeting. Near the east end of the viaduct there was considerable old, filled material; and rock was encountered at various elevations. The east abutment rests in part on clay and shale with front toe-walls, 5 ft. thick, carried to rock at about 10 ft. lower elevation. The work was simple, open excavation.

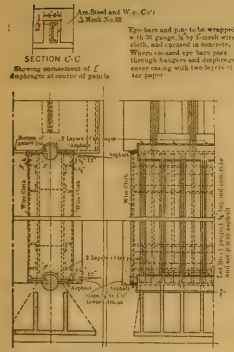
After excavations were made, the Raymond concrete piles were driven, in the usual manner, to the refusal the apparatus would stand. The engineers required that all the steel shells for a group of piles be driven and then the concrete for all be placed at one time. This was to prevent any possible damage to the unset concrete of one pile by the driving of another near it. No particular difficulty resulted, except that the shells would sometimes partly fill with water or with sand and silt, which had to be pumped out before depositing the concrete. The shells stood open without collapse, except where water-bearing, so-called "quicksand" was encountered, and there it was found necessary to fill each shell as driven. In the main, the concrete for the piles was mixed in the general mixing plant, but, in some instances, for convenience, small quantities were mixed by hand.

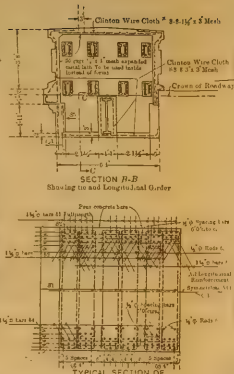
Forms and Falsework.—The forms and falsework merit special attention, for their construction, erection, and removal formed the principal items of labor cost of the whole structure, and the efficiency with which the work was carried forward was dependent in no small degree on the thoroughness and care the contractor used in their design and construction.

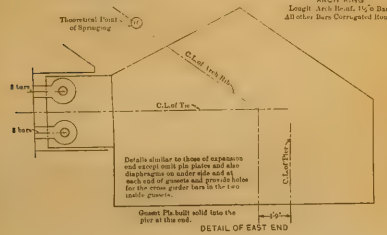
Plans and sketches were prepared dimensioning all special parts of all forms, and, as far as possible, controlling portions were constructed in the carpenter shops and taken out entire for erection; only the principal portion of the sheathing was put on in place in the usual manner. Though the structure has an effect of duplication, there is, in reality, very considerable variation of dimensions within small limits, so that there were few parts of forms removable and replaceable without alteration. For instance, there are no two columns of the entire structure alike.

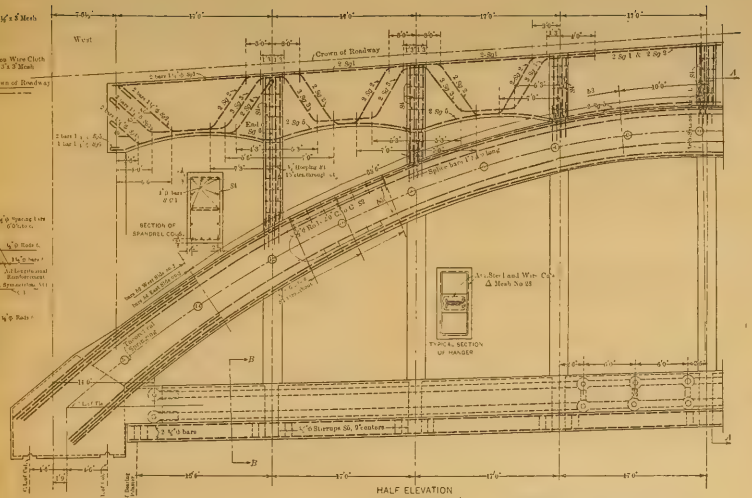
The forms for the columns were built to dimensions in the carpenter shops in three principal parts; a ready-to-erect form for plinth and for capital, and partly finished forms for the shafts, comprising four side frames of correct height and batter, from 8 to 12 in. wide at each corner, or rather extending back to the raised panel of

PLATE XVI.
PAPERS, AM SOC C. E.
MAY, 1915.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.









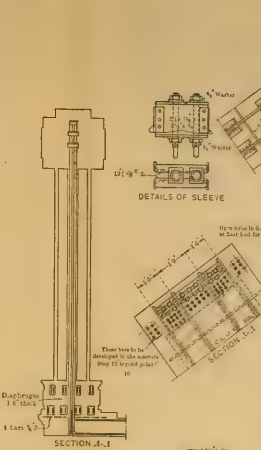
GENERAL NOTES.

Except where otherwise noted the minimum spacing shall be 3 diameters of the bar
" " " " " edge distance shall not be less than 2 dia. of the bar. (℄ L of bar to edge of concrete)
" " " " " bars shall be bent to a radius of not less than B dia.
" " " " " hooks must be of dimensions as shown in this sketch

$R = 2\frac{1}{2}$ diameters of the bar $A = 3$ diameters of the bar

For splicing, the bars must be overlapped 40 diameters, and wired securely together
The splices must be alternated, so that in no case more than half the number of bars are spliced at the same point. Where possible the splices should be located at points of minimum tension

NOTE
The ties are to be shortened 1.16′′ after being made taut, measured between toggle pins as indicated in sketches or such an amount that the distance between springing points, after dead load, is on the arch, ½ % shorter than constructed
Rivets ⅞ Ø — All holes sub punched and reamed to 15/16


Structural Steel ℄ L
SECTION J-K
1" Washer
1" Washer
DETAILS OF SLEEVE
Up-set holes in Gusset Pls at East end for 3" rods
Bars 1 of 7
Bars 1 of 7
Pin Pls. 13" x
Pin Pls. 13" x
Pins 6½ x
Pins 6½ x

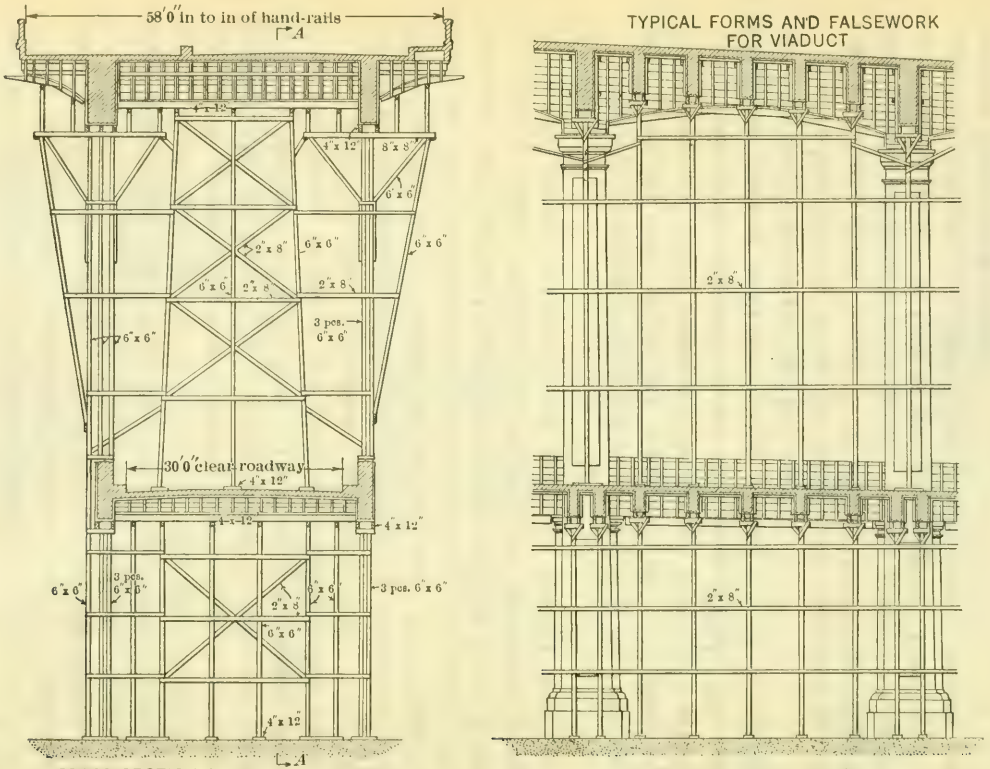

58′0″ in to in of hand-rails
A
4″x 12″
8″x 8″
6″x 6″
6″x 6″
2″x 8″
6″x 6″
6″x 6″
2″x 8″
2″x 8″
3 pcs. 6″x 6″
6″x 6″
30′0″ clear roadway
4″x 12″
4″x 12″
4″x 12″
6″x 6″
3 pcs. 6″x 6″
6″x 6″
2″x 8″
3 pcs. 6″x 6″
6″x 6″
4″x 12″
A
CROSS-SECTION SHOWING FALSEWORK
TYPICAL FORMS AND FALSEWORK FOR VIADUCT
2″x 8″
2″x 8″
LONGITUDINAL SECTION A-A

FIG. 13.

the column. The sheathing over the panels was prepared and placed after the side frames were adjusted to position. Yokes surrounding the column of the usual four timbers bolted at the corners, and tied across with a central bolt surrounded by a tin tube, were placed at intervals and given bearing by wooden wedges.

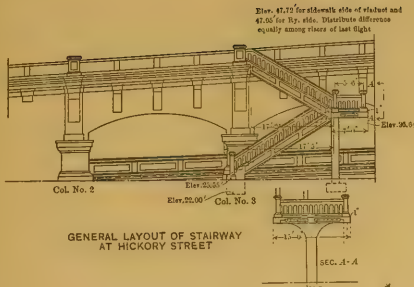
For the upper-deck longitudinal girders, and the cross-girders between columns, which had curved bottoms, the forms were built in place by the usual building-construction methods. The curved joists under these girder forms were framed and bolted together full length, carried out, and placed entire; and short lagging boards, the length of the girder width, were nailed on. The side-lagging was placed longitudinally on 2 by 6-in. studding, placed in pairs at suitable intervals, and tied across by bolts. The lower bolts passed through below the bottom sheathing, and were not encased by the concrete.

The forms for the intermediate cross-beams were of the same general design, except that the sides were made of a series of panels about 2 ft. wide with a vertical stud at each side. After the bottom forms had been prepared, these panels were easily and rapidly set up alongside each other and supported by bolts passing across between the pairs of studs. The erection, removal, and re-erection of these panels was simple and easy; and their use effected considerable economies.

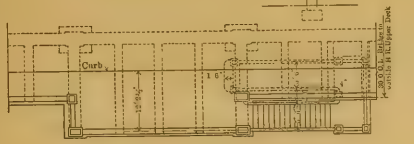
The cantilever beams on each side of the structure were also duplicated forms and were built in two unit-sides ready to be placed on a bottom form and bolted together. The bottom joists were sawed to correct form in the shop and sheeted after being placed. The cracks between the sheeting on all these curved surfaces were filled, or pointed up, with clay just before placing the concrete.

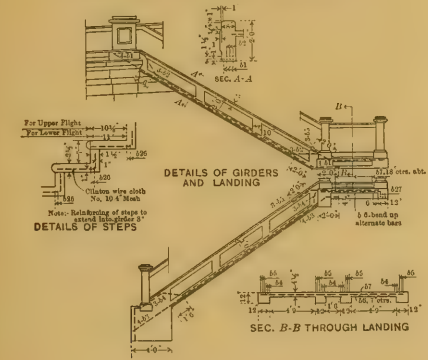
The decking between beams and girders was supported on joists resting on string pieces fastened to the vertical studs of the beams and girders, as is customary in building construction. Forms for abutments and retaining walls were built in the usual fashion. Portions of the hand-rails, stairways, etc., were built of units cast in wooden forms generally lined with sheet steel.

The falsework supporting the lower-deck forms consisted of bents of posts under each cross-girder, braced together transversely and longitudinally. A short longitudinal cap was placed on each post with small diagonal braces. The bents were built complete with


Elev. 47.72 for sidewalk side of viaduct and 47.05 for Ry. side. Distribute difference equally among risers of last flight
Elev.36.04'
Col. No. 2
Elev.23.55
Elev.22.00
Col. No. 3
SEC. A-A

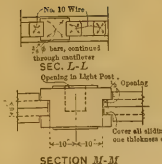
GENERAL LAYOUT OF STAIRWAY AT HICKORY STREET



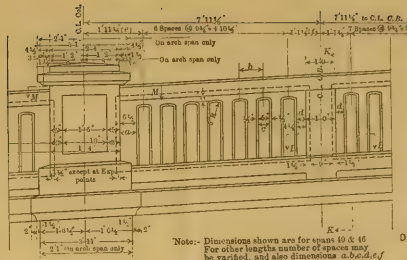

SEC. A-A
For Upper Flight
For Lower Flight
Clinton wire cloth No. 10 4 Mesh
Note:- Reinforcing of steps to extend into girder 5"
SEC. B-B THROUGH LANDING

DETAILS OF STEPS

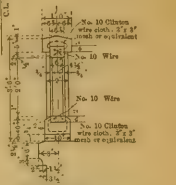
DETAILS OF GIRDERS AND LANDING


No. 10 Wire
½"ø bars, continued through cantilever
SEC. L-L
Opening in Light Post
Opening
Cover all sliding surfaces with one thickness of Tar Paper

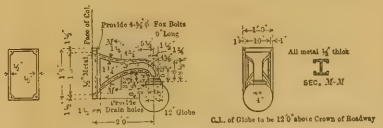
SECTION M-M
SHOWING EXPANSION JOINT FOR HAND-RAIL



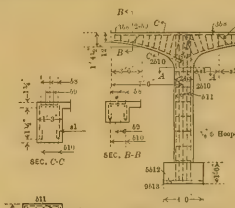
Note:- Dimensions shown are for spans 40 & 46
For other lengths number of spaces may be varied, and also dimensions a,b,c,d,e,f
Maximum variation of any dimension, 1"

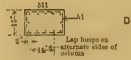


SECTION K-K
DETAILS OF UPPER DECK HAND-RAIL AND FASCIA GIRDER

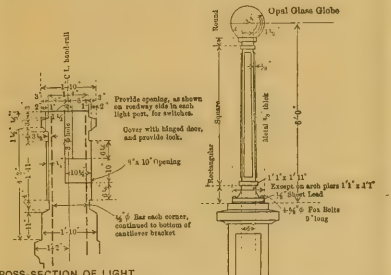

Provide 4-¾"ø Fox Bolts 9' Long
Face of Col.
Metal
Provide 1½ cm Drain holes
12" Globe
All metal ½" thick
SEC. M-M
C.L. of Globe to be 12'0" above Crown of Roadway

DETAIL OF CAST-IRON LAMP BRACKET FOR LOWER DECK)





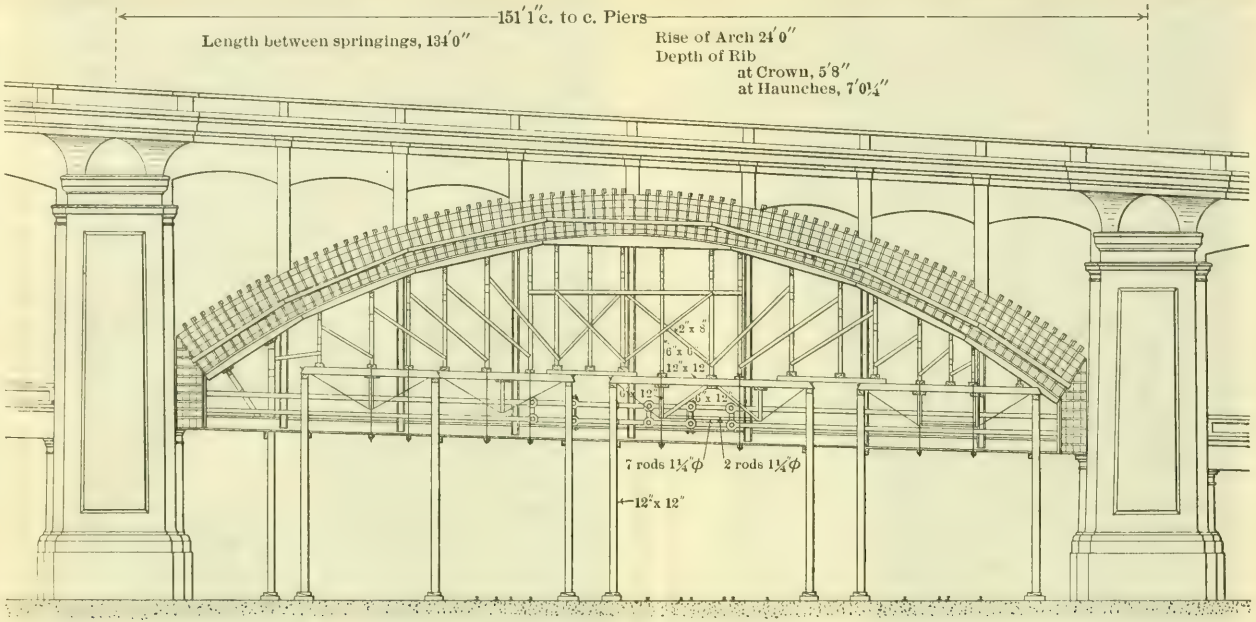
DETAILS OF COLUMN


Opal Glass Globe
Round
Square
Rectangular
Metal ⅛ thick
Except on arch piers 1'1" x 1'1"
Fox Bolts 9" long
C.L. hand-rail
Provide opening, as shown on roadway side in each light post, for switches.
Cover with hinged door, and provide lock.
4"x 10" Opening
½"ø Bar each corner, continued to bottom of cantilever bracket

CROSS-SECTION OF LIGHT POST (AT C.L.)
OTHER MAIN POSTS SIMILAR, BUT WITHOUT OPENINGS.

DETAIL OF CAST-IRON LAMP STANDARD UPPER DECK

DETAILS OF
HAND-RAILS, STAIRWAYS,
LIGHT BRACKETS. ETC.
TWELFTH STREET TRAFFICWAY VIADUCT
KANSAS CITY, MO.



ELEVATION OF ARCH SPAN
SHOWING
FALSEWORK FOR ARCH RIB

FIG. 14.

bracing on the ground and set up with the derrick traveling on the lower deck. The posts were generally 6 by 6-in. timbers, supported on ordinary ground-sills, and the bracing was of 3 by 6-in. pieces bolted in place.

The falsework supporting the upper deck forms was of similar bents resting on the lower-deck floor-slabs, with the posts under the main girder supported on the main girders below. These were made in three parts; a central bent of three posts and bracing supporting the central portion of the cross-beam, and two side bents of three posts each with a cap cantilevered to each side, one end of the cap supporting the cantilever beam and the other extending far enough under the cross-beam to carry a corresponding load, balancing the entire cap load over the posts. These three parts of each bent were prepared on the lower deck near position, or were carted out to position on wagon trucks and raised by the derrick on the upper deck.

The requirements of horizontal clearance for the railroad tracks on Santa Fé Street made necessary the use of three pairs of small wooden truss spans to support the forms for the arch ribs. These trusses rested on bents of posts set in available spaces between the tracks. The arch centering was placed on top of them, and the lower deck forms were suspended from them.

Falsework and forms for about seven girder spans were provided and carried forward as it was removed, the posts of the falsework being spliced as the height increased.

Procedure.—The falsework, forms, and pouring were each, relatively, on the lower deck about two span lengths ahead of the upper deck, as the falsework for the upper deck could not be placed on the lower-deck concrete until it had hardened somewhat. The whole construction thus moved forward in sequence, maintaining about the same relation of lower deck advance. This sequence of operations is shown by Fig. 20.

The reinforcing bars were delivered, as far as possible, in lengths correct for use. Diagrams were prepared showing the shape and dimensions for bending each bar and assigning a mark to it. The bars were bent in the yard, and each special bar was tagged with its erection mark on a linen tag. Groups of plain bars, etc., were not tagged individually.

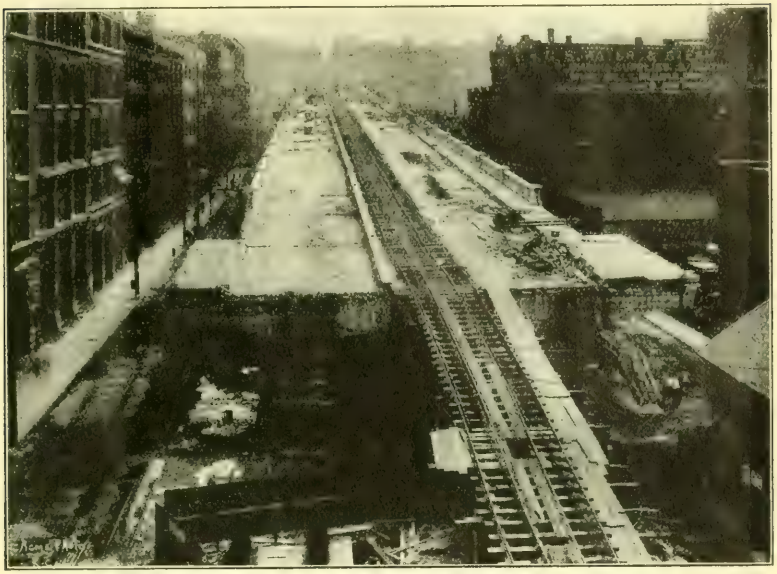


FIG. 15.—UPPER DECK, TWELFTH STREET TRAFFICWAY VIADUCT, DURING CONSTRUCTION.

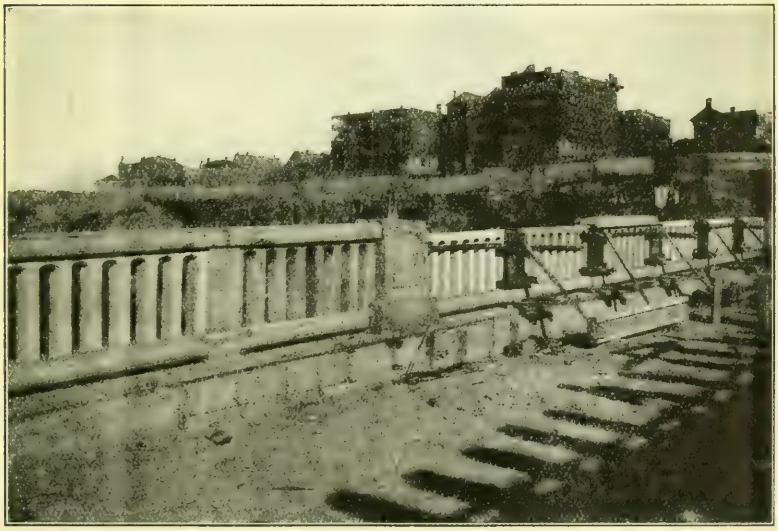


FIG. 16.—METHODS OF BUILDING HAND-RAIL, TWELFTH STREET TRAFFICWAY VIADUCT.

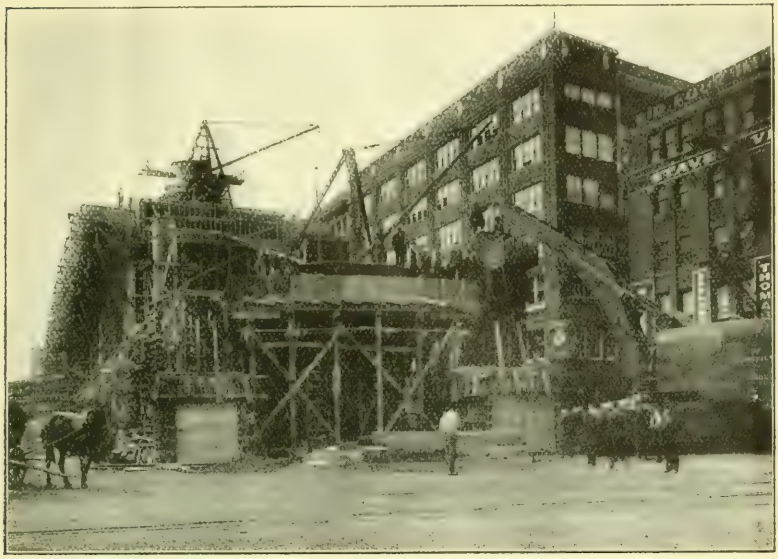


FIG. 17.—SANTA FÉ ARCH SPAN: ROCKER BENTS AND ARCH-RIB GUSSET-PLATE, TWELFTH STREET TRAFFICWAY VIADUCT.

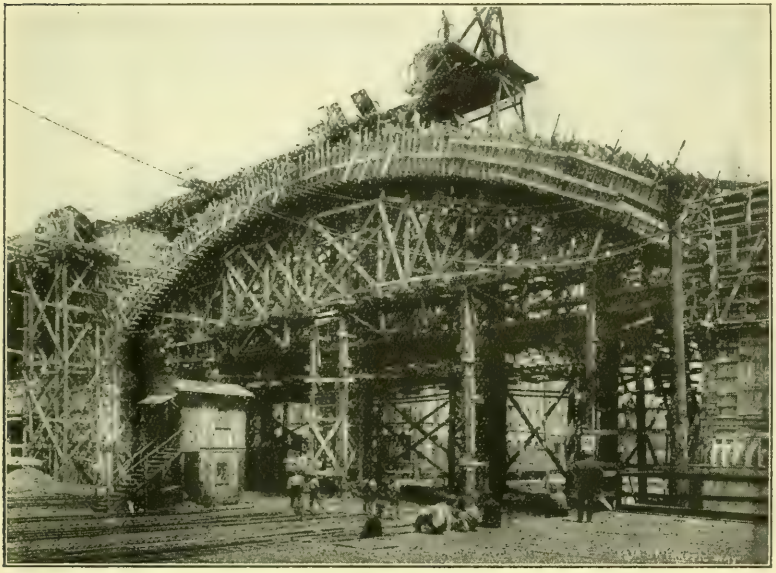


FIG. 18.—FALSEWORK AND FORMS, SANTA FÉ ARCH SPAN, TWELFTH STREET TRAFFICWAY VIADUCT.

FIG. 19.—LOWER DECK ADJACENT TO ARCH SPAN, TWELFTH STREET TRAFFICWAY VIADUCT, DURING CONSTRUCTION.

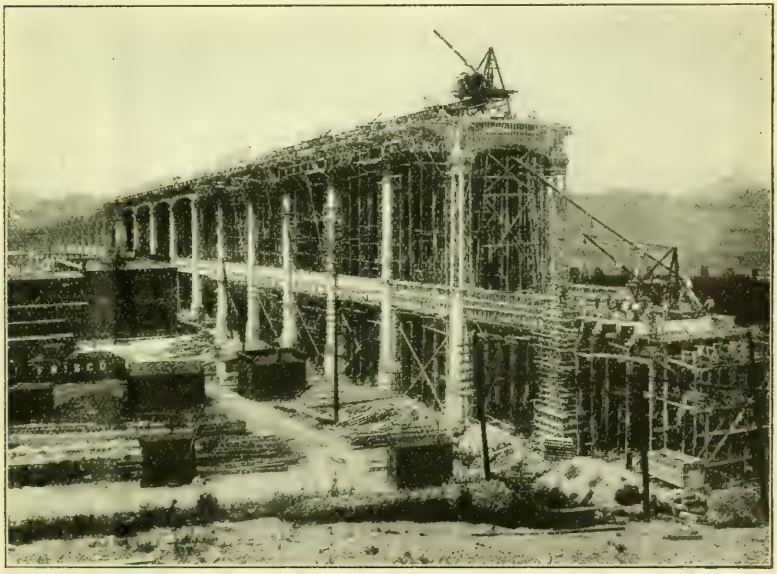


FIG. 20.—TWELFTH STREET TRAFFICWAY VIADUCT: SEQUENCE OF OPERATIONS DURING CONSTRUCTION.

As the forms neared completion, the prepared bars were carried out on the tram cars and set in place. Metal clips, separators, and wire ties at the crossing of every two bars, supported them in place. Use was made of small cement briquettes or blocks in some cases to hold the bars up from the forms.

The placing of bars and depositing of concrete in the columns was carried forward in customary routine. For the upper deck, all the bars were placed for main girders, cross-girders, cantilevers, and slabs complete from the last division to a bulkhead placed across the structure at a convenient point. Concrete was poured continuously across the whole width of the structure, filling the girders, beams, slabs, etc., without a stop, for the longitudinal distance selected.

The concrete was transported in cars, as described, hauled by cable, discharged into a movable distribution hopper, and from this into Insley chutes, which conveyed it to final position in the forms. A trussed steel chute about 80 ft. long carried the concrete from the receiving hopper on the upper deck forward to position in the lower portions of the structure. The concrete was mixed to a soft consistency so that it flowed readily, but it was not so wet as to have water standing on its surface. It was spaded and sliced at the forms and around the reinforcing bars to insure complete contact, and to work forward the mortar.

The maximum capacity of the concrete plant delivering 1 300 ft.—or, say, to the mid-point of the structure—was about 30 cu. yd. per hour. The largest quantity placed in continuous run was about 450 cu. yd. in 15 hours. A trip one way, from the plant to the east abutment, required 3½ min.

The hand-rail units were cast on the deck in metal-lined wooden forms, near the place where they were to be used. The lower rail was cast in place, the separately cast balusters were set in it in rows, and the pre-cast top rail was set on them, extending into the posts. The forms for the posts, except for the capital, were clamped on, and the posts were cast in place. The stairs were cast in units of one riser and one tread. These were supported on falsework and projected into the forms for the supporting girders, which were built in place.

Materials and Wages, General Observations.—The total quantity of timber used on the job was about 1 500 000 ft. b.m. This may be

roughly divided into 600 000 ft. of form lumber, 850 000 ft. of falsework, and 50 000 ft. in general use, including bins and mixing plant. The average cost of this material was about $20 per thousand or about $1 per yd. applied to the whole concrete, or $1.25 per yd. applied to concrete above the bases.

The number of men employed varied from 150 to about 325, an average of about 200 men, of whom, perhaps, 150 were carpenters and helpers. The wages prevailing were, for common labor, $2.00, for carpenters' helpers, $2.50, for carpenters, $3.50, for finishers, $3.50, and for derrick and hoist operators, $4.00; all for 9 hours per day.

The conduct and execution of the work merit special comment. The viaduct was built substantially in the allowed time of 15 months, moving forward in logical and orderly manner, largely due, of course, to systematic handling of labor and materials. On the first day of every week, a schedule of work to be accomplished each day of that week was prepared in writing, and the foremen were instructed, not only what to do each day, but also how much had to be done. If the amount assigned was not completed at the usual stopping time, the men kept at work until it was done. This applied particularly to placing concrete, the day's run being completed more than once after midnight. The conditions surrounding the work permitted it to be handled as a manufacturing proposition, and with factory-like precision and system.

There was some difficulty in securing stone as clean as desired, for the limestone strata in this region are overlaid with clay and other objectionable materials; and, especially during a rainy season, it is impossible to prevent some clay and soft shale from mingling with the stone in the crusher bins. In fact, the more careful crushers shut down during rainy weather to avoid this difficulty. The stone cost $1.15 per cu. yd., delivered in cars at the site. The sand supply from the Kaw River is clean, sharp, and well graded, testing about 52% retained on a No. 30 sieve. Its cost delivered in cars at the site was about 65 cents per cu. yd. The cement was Kansas City-Portland, manufactured in a mill about 15 miles from the site, and costing, delivered in cars at the site, $1.20 per bbl. (plus sack loss). These delivered prices include somewhat complex switching charges.

Reinforcing steel delivered cost about 1.75 cents per lb.

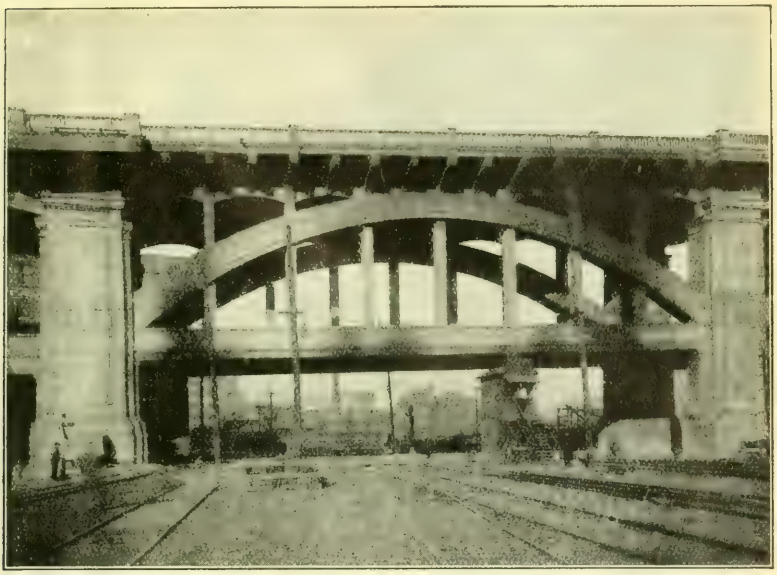


FIG. 21.—SANTA FÉ ARCH SPAN AFTER COMPLETION, TWELFTH STREET TRAFFICWAY VIADUCT.

FIG. 22.—TWELFTH STREET TRAFFICWAY VIADUCT, LOOKING EASTWARD ON THE LOWER DECK.

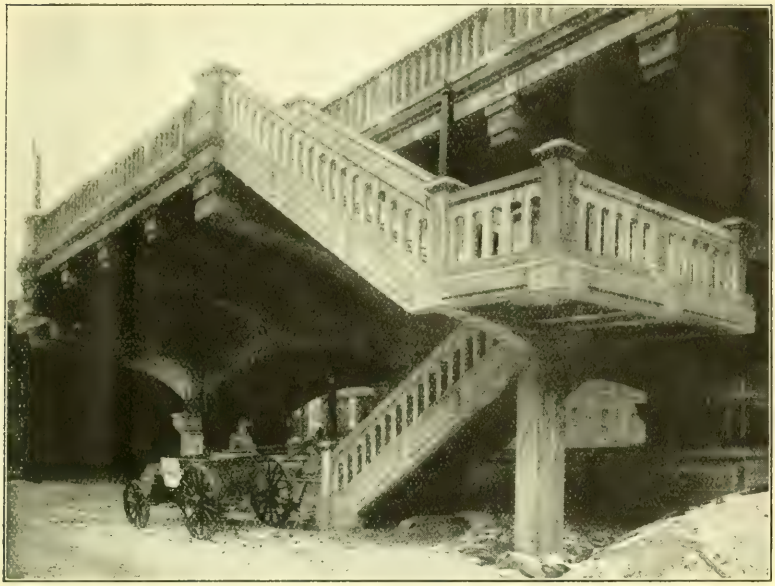


FIG. 23.—STAIRWAY AT HICKORY STREET, TWELFTH STREET TRAFFICWAY VIADUCT.

FIG. 24.—STAIRWAY AT MULBERRY STREET, TWELFTH STREET TRAFFICWAY VIADUCT.

6.—Quantities and Costs.

The contract assigns unit prices for forty classified items, including many classifications of concrete with regard to location in the structure. The following summarized figures were prepared from such classifications, and an attempt has been made to give the salient features of the final quantities of the structure, as built. These costs are the amounts paid by the City, and include the contractor's profit.

There are about 32 500 cu. yd. of concrete, distributed about as follows:

	Cubic yards.
Concrete piles	1 360
Columns, piers at Santa Fé Street, and footings	11 300
Upper deck, including girders, cross-girders, cantilevers, floor-slabs, etc.	11 300
Lower deck, including girders, cross-girders, floor-slabs, etc.	3 600
Retaining walls and abutments	3 000
Ribs of arch at Santa Fé Street	600
Hand-rails	500
Stairways, sidewalks, curbs, paving base on fills, and miscellaneous	800

No special payment was made for excavation, but the amount paid for concrete included its cost. The total excavation was about 10 000 cu. yd. The amounts paid for concrete in place varied from about $8.00 to $13.50 per cu. yd. for different classifications, averaging $9.45 for the whole structure, omitting only the piles and the hand-rails, as both of these were paid for at linear foot prices.

There were driven 2 610 piles, having a total length below bottoms of footings of about 47 000 lin. ft. and an average length each of 18 ft., costing in place $1.20 per lin. ft.

The approximate quantities of metal, and the prices paid for it in place are as follows:

Reinforcing steel, $\frac{3}{4}$ in. in diameter and greater	2 900 000 lb. at 2.95 cents per lb.
Reinforcing steel, less than $\frac{3}{4}$ in. in diameter	1 000 000 lb. at 3.25 cents per lb.
Unfabricated metal, such as bed and bearing plates, guard-angles, etc.	100 000 lb. at 4.40 cents per lb.
Fabricated structural metal	620 000 lb. at 4.60 cents per lb.
Cast-iron lamp-posts and brackets	25 000 lb. at 5 cents per lb.

Omitting the fabricated structural metal and the lamp-posts gives a total of 4 000 000 lb. of metal, or, considering 30 600 cu. yd. of concrete, an average of 130 lb. of metal per cubic yard, at an average price of 3.06 cents per lb.

The lighting system, complete as described, cost $9 500.

The double street-car tracks, including ballast, cross-ties, track rails, inner steel guard-rails, bonding, and trolley poles, cost, in place, a total of $18 000 for a length of 2 600 ft. of double track, or, say, $3.50 per lin. ft. for single track.

There are 14 500 sq. yd. of wood-block paving, which cost $2.25 per sq. yd. The water-proofing under the ballast comprised about 5 000 sq. yd., costing $1.71 per sq. yd., and that under the pavement included 11 500 sq. yd. at a cost of 58.5 cents per sq. yd.—a total cost of $15 000.

There were some 40 000 cu. yd. of earth filling, costing 28 cents per cu. yd.

There were other miscellaneous items such as catch-basins, drainage pipes, sewer connections, etc., bringing the total cost of the structure to the city to about $600 000, to which should be added some $40 000 for engineering.

The following is a summarized table of quantities and costs:

Concrete piles (2 610)	$60 000
Excavation, about 10 000 cu. yd.	(no price)
Concrete, 30 600 cu. yd.	290 000
Reinforcing metal, 4 000 000 lb.	122 000
Structural metal and castings, 645 000 lb.	30 000
Lighting system	10 000
Street-car tracks, etc.	18 000
Water-proofing	15 000
Block pavement	33 000
Hand-rails	8 000
Earth filling and miscellaneous	14 000
	$600 000

7.—Personnel.

The Twelfth Street Trafficway Viaduct was built by the City of Kansas City, Mo., under control of the Board of Public Works. The plans were made and the construction supervised by Messrs. Waddell

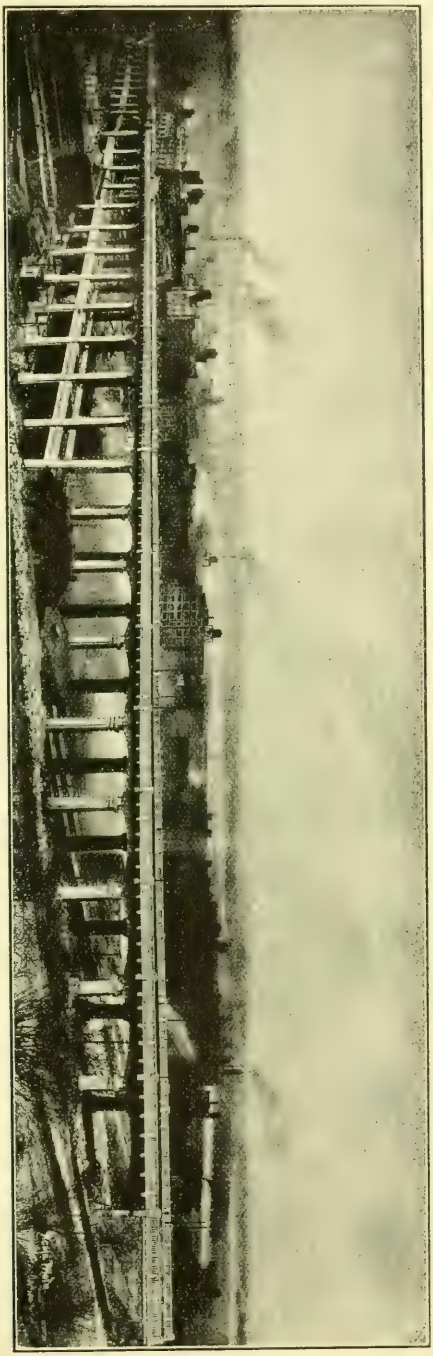


FIG. 25.—TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MO.

and Harrington, Consulting Engineers, under the personal supervision of Mr. Harrington. The writer prepared the general preliminary studies and estimates, directed the design, and wrote the specifications. L. R. Ash, M. Am. Soc. C. E., was City Engineer at the time the plans were adopted, and after joining the firm, directed the construction work. The details of design were largely the work of Messrs. A. E. Slettum and C. W. Yelm. Mr. M. J. Maher was Resident Engineer, in charge of the entire construction work.

The structure was built under one general contract by The Graff Construction Company, of Seattle and Kansas City. Mr. Graff gave the work his constant personal attention, and was ably assisted by his superintendent, Mr. Harold E. Ketchum, in handling the building operations and in the design of the contractor's plant, falsework, forms, and other temporary structures; Mr. George Hockensmith was General Foreman.

For their courteous assistance in the preparation of these data, the writer desires to express his cordial appreciation to the above gentlemen and to N. C. Tammen, Assoc. M. Am. Soc. C. E., Designing Engineer of Harrington, Howard, and Ash.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE PICAZA BRIDGE*

By A. A. Agramonte, Assoc. M. Am. Soc. C. E.

Synopsis.

This paper contains a description of a reinforced concrete bridge spanning a part of the Valley of the Salada River, in the Province of Buenos Aires, Argentine Republic, and, together with ten other bridges and some smaller works, was built under the direction of the Drainage Commission of the Province of Buenos Aires.

The bridge consists of a central span of 26 m. (85 ft.) and six lateral spans of 28.5 m. (93.5 ft.) each. The central span consists of a floor system suspended by steel tension members from two concrete arches each of which is reinforced with a structural steel skeleton arch, and tied by a steel chord to take up the thrust. The hangers are bolted to the chord, and all the tension members are encased in concrete. Roller bearings at one end provide for expansion and contraction. Two transverse arches supply wind bracing and help to make the structure rigid.

Each lateral span consists of three continuous reinforced concrete beams resting on three supports. The central supports are concrete pylons, and are fixed. The end supports are masonry pillars, and steel bearing plates, on pillars and beams, face to face, allow free tempera-

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

ture movement. The floor and sidewalk are carried on a transverse beam and slab system.

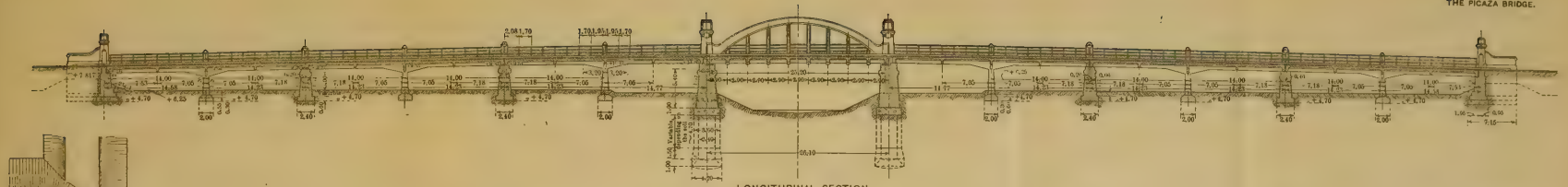
A brief description is given of the drainage works and the structures involved, and then the Picaza Bridge is described, the working organization, the foundations, the materials, the labor, the finish, the progress, and the costs.

General Description of the Drainage Works.

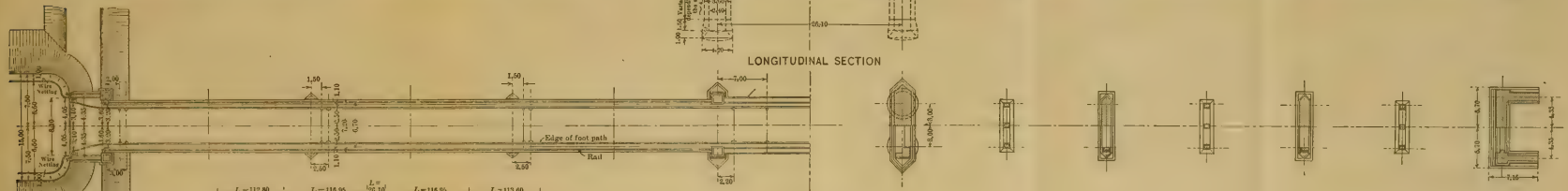
The Region.—The Drainage Commission of the Province of Buenos Aires is a board of land owners having charge of the design and execution of works for the relief of the southern part of that Province, a region subject to inundation at intervals of from 10 to 15 years. This region is a great plain dipping gradually toward the Atlantic Ocean on the east, and receiving the discharge of the flashy hill streams, or "arroyos", which drain the uplands and "sierras" on the west. Below this upland region watercourses disappear from lack of fall, and, in the whole area subject to inundation, there is only one river, the "Rio Salada", with a continuous channel, and this is utterly inadequate to convey the flood flow from its own water-shed. The area subject to inundation is approximately 39 750 sq. km., or more than 9 700 000 acres.

The Project.—To ameliorate the disastrous effects of the periodical inundations in a country with no natural drainage channels, a number of canals have been projected and built for the double purpose of, first, carrying the average flood flows of the "arroyos" directly to the sea at a relatively high level, and secondly, receiving the drainage of the lower zones through numerous sluice-gates at a low level after the discharge of the flood flow of the "arroyos". Complete flood prevention would be economically impossible. The system takes a normal flow of 46 000 000 cu. m. per day (18 800 cu. ft. per sec.).

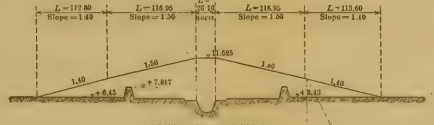
These canals are of many sections and lengths, but a description of them would be foreign to the object of this paper. It is sufficient to state that they consist in general of a central excavated section from 2 to 4 m. ($7\frac{1}{2}$ to 15 ft.) deep and from 10 to 30 m. (33 to 108 ft.) wide and a flood section formed by lateral dikes from 1.5 to 4 m. (5 to 13 ft.) above the ground level, and from 80 to 300 m. (262 to



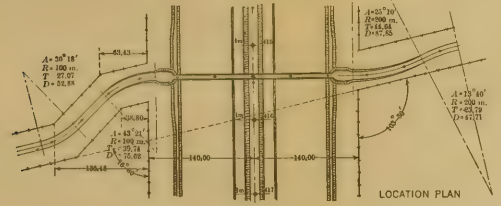
LONGITUDINAL SECTION



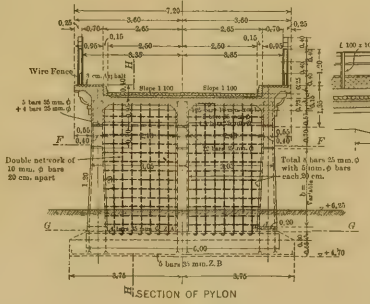
PLAN



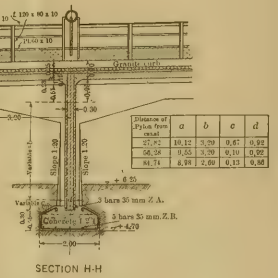
LONGITUDINAL SECTION



LOCATION PLAN



SECTION OF PYLON



Distance of Pylon from canal	a	b	c	d
27,82	10,12	3,70	0,67	0,92
56,54	9,55	3,70	0,10	0,92
81,74	8,98	2,00	0,13	0,95

SECTION H-H

GENERAL PLAN
PICAZA BRIDGE
OF THE
DRAINAGE COMMISSION
OF THE
PROVINCE OF BEUNOS AIRES
ARGENTINA

984 ft.) in effective width between the dikes. Three of the canals are more than 130 km. (80 miles) long, one is more than 260 km. (160 miles) long, and there are sixteen others of varying lengths, and thousands of miles of laterals. The total length of the main canals is 1 005 km. (624 miles) and the contents are more than 30 000 000 cu. m. (39 240 000 cu. yd.). The total cost to date has been $40 000 000 of official paper money, equivalent to more than $17 400 000 in gold.

Control and Accessory Works.—To discharge flood waters under control, and conserve the natural watercourses where convenient, structures of the following types were built:

1.—The works at the entrance of the derivation canals consisted of entrance gates controlled by machinery, and waste-weirs or gates (generally automatic) to discharge the excess flow to natural channels. The gates and appurtenances are of steel and bronze; the structures are of reinforced concrete with steel skeletons, hoop-bound brickwork or hardwood ("quebracho"), according to location. Piling and sheet-piling were used where necessary. Works of this type have been constructed at the origin of nine canals.

2.—Sluice or drainage gates were placed at appropriate places in the length of the canals, and provide for the drainage of the low-level zone. The gates and appurtenances are of steel and bronze, but hardwood gates are also used. All types are automatic. There are 114 of these structures.

3.—Relief openings are provided with gates operated by machinery and serve to supply or to empty large "lagunas" or natural reservoirs. The gates, etc., are of metal and the structures are mainly of reinforced concrete. There are five of these openings.

4.—Waste-weirs take the water from the canals to natural watercourses, at places of strategic importance. These are automatic, and generally of reinforced concrete or hoop-bound brickwork. There are six of these weirs, exclusive of those at the head-works.

5.—Inverted siphons are used to pass low-level streams under the canal where it has been found necessary to maintain such channels for the drainage of certain districts. There are three of these.

6.—Bridges have been built wherever traffic has required them. They are of varying spans, types, and materials. Table 1 gives information regarding all the bridges built.

TABLE 1.—Bridges Built by the Drainage

Canal No.	Distance from mouth, in kilometers.	Distance to cart materials, in kilometers.	Foundations.	
			Abutments.	Pillars.
1	30	40	Hardwood piles	Steel cylinders
1	65	15	" "	Concrete cylinders
1	106	40	Slab concrete	Slab concrete
2	4	4	Hardwood piles	Steel cylinders
2	36	36	Slab concrete	Slab concrete
2	62	15	" "	" "
2	88	15	" "	" "
2	107	20	" "	" "
3	11	20	Steel piles	
5	6	30	Cylinders, reinforced concrete	
5	34	35	Slab, reinforced concrete	Cylinder. Reinforced concrete
9	4.6	35	Hardwood piles	
9	41	5	Slab, reinforced concrete	Cylinder. Reinforced concrete
9	81	40	Slab concrete	Slab concrete
9	102	30	" "	" "
9	141	25	" "	" "
10	....	35		
15	11	30	Cylinders	Screw piles
15	33	15	Caissons	
18	6	45	Hardwood piles	Hardwood piles
18	17	35	Reinforced concrete cylinders	
24	4	4	Hardwood piles	
24	37	37	" "	

Note: Ten more bridges are to be built. * Brick plastered

The Picaza Bridge.

Location.—The Picaza Bridge, 5 km. (3 miles) from the City of Dolores on the highway to Sevigné, in common with all the works described, was designed by the Technical Office under the supervision of the late Engineer Director, Mr. Charles Nyströmer. The execution of the work, together with that of ten other bridges and minor structures, was assigned to the writer under Mr. Humphrey W. Cooper, Chief Engineer. The Inspector General of all the works of the Drainage Commission was Mr. Leonard Cooper.

General Description.—The bridge is composed of one central span of 26 m. (85 ft.) and six lateral spans of 28.5 m. (93.5 ft.) each, all in reinforced concrete. The central span consists of a floor system suspended by steel tension members from two concrete arches each of which is reinforced with a structural steel skeleton arch and tied by a steel chord to take up the thrust. The hangers are bolted to the chord, and all the tension members are encased in concrete. Roller bearings at one end enable the arch to move with variations of tem-

COMMISSION OF THE PROVINCE OF BUENOS AIRES.

ELEVATION.		Superstructure.	Span (over all), in meters.
Abutments.	Pillars.		
Stone.....	Steel and concrete.....	Steel cantilever. 3 openings	114.80
"	Stone.....	" " "	90.00
"	"	" " "	73.50
Brick, plastered *....	Brick, plastered *.....	" " "	73.50
" "	" "	" " "	145.08
" "	" "	" " "	145.08
" "	" "	Steel truss.....	97.40
" "	" "	" "	97.40
Reinforced concrete..	.....	Steel trestle.....	37.10
Brick, plastered *....	.....	Steel truss, one span.....	44.00
" "	Brick, plastered *.....	Steel cantilever. 3 openings..	145.00
" "	" "	One steel truss, two of reinforced concrete.....	72.40
" "	Brick, plastered, and reinforced concrete.....	Reinforced concrete viaduct..	197.50
" "	Brick, plastered *.....	Steel cantilever. 3 openings..	143.90
" "	" "	" " "	143.90
Stone.....	Stone.....	" " "	114.80
Steel.....	.....	Steel trestle.....	.....
Brick, plastered *....	Brick, plastered, and steel	Steel trestle and truss.....	145.50
" "	Brick, plastered, and reinforced concrete.....	Steel cantilever. 3 openings..	145.10
Hardwood.....	Hardwood.....	Hardwood.....	15.00
Reinforced concrete..	.....	Reinforced concrete. 1 span.	13.00
Brick, plastered *....	.....	3 spans. Reinforced concrete	18.20
" "	.....	" " "	14.20

to imitate stone. † On branch canals.

perature. Two transverse arches provide for wind bracing and help to give complete rigidity to the structure.

Each lateral span consists of three longitudinal continuous reinforced concrete beams resting on three supports; the central support is a concrete pylon, and is fixed. The end supports are masonry pillars. Steel bearing plates on pillars and beams, face to face, allow free temperature movements. A transverse beam and slab system carries the floor and sidewalk.

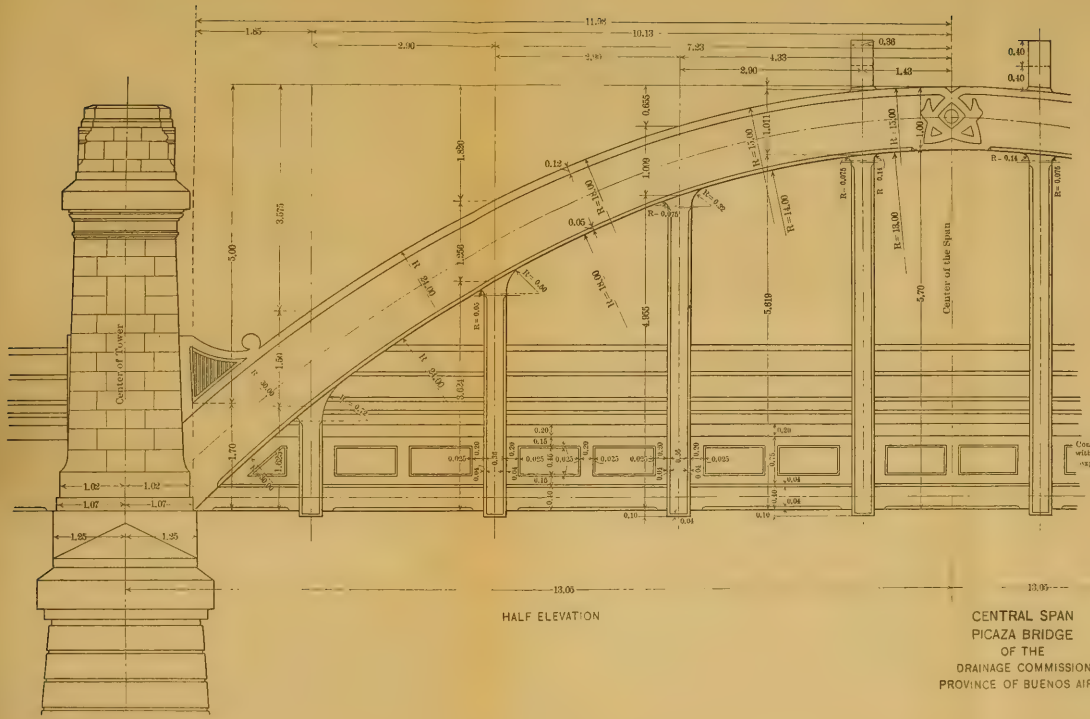
Organization.—As far as possible, the work was done by using both piecework and bonus systems. Ordinary day labor was used where unavoidable on a very small proportion of the whole. The writer, although having under construction many other bridges at the same time, made a point of directing all important work personally, a difficulty which may be realized when it is stated that it required a two days' trip to reach some of the other bridges. A general foreman and a time and storekeeper completed the permanent force in charge of the work.

Those who examine the statement of costs of this bridge should bear in mind that the general foreman had to be instructed in his duties while on the work, as no sober, experienced, and reliable concrete men having the requisite knowledge and skill were to be found in Buenos Aires when the work was commenced. Further, laborers in summer desert the work to go to harvesting, with the expectation of higher wages. Form carpenters, also, have to be taught on the work, and, as they know that new men do not acquire the necessary experience in a short time, they soon become very urgent in their demands for increased wages. In brief, reinforced concrete construction was a continual struggle, first to teach the men and afterward to hold them on the work, and this was aggravated by the difficulty of obtaining reliability in foremen and timekeepers, a greater problem than it may seem from a distant viewpoint.

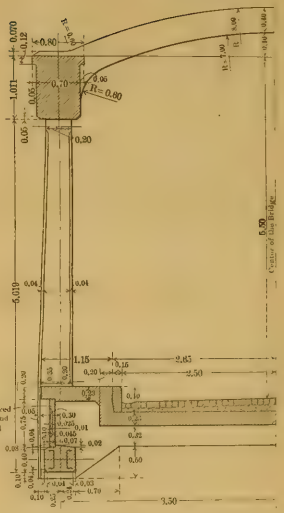
For the foregoing reasons labor costs should be higher than in regions more favored in the respects indicated; nevertheless, the writer may claim that they represent fair costs for that part of the country, as the work was done for 30% less than the lowest prices bid for it by a contracting firm, and 50% less than the next highest bid (including all administrative charges, etc.). The costs given may be taken as representative for work of this class in the Province of Buenos Aires at the present time.

Excavation of Foundations.—The excavation was in hard clay, which was shoveled into Decauville cars and transported by horse traction about 200 m. (650 ft.) to the lateral dikes of the canal. The spoil from the cylinder excavation followed the same route.

Foundations.—The foundations of the lateral piers and abutments were of 1:3:5 concrete, with crusher screenings instead of sand. The materials were brought from stock piles about 100 m. (328 ft.) from the axis of the bridge, in Decauville cars to a portable mixing platform placed as near the work as possible. The stone and screenings were measured in the cars. The concrete was mixed by hand and placed directly from the platform. Two mixing platforms were used, and were moved as needed. The stone had to be washed well before it was used, as it had been stored in an exposed position for more than a year, and was full of dirt. Water was provided by a windmill pumping to two tanks having a capacity of 12 cu. m.



HALF ELEVATION



HALF TRANSVERSE SECTION

CENTRAL SPAN
PICAZA BRIDGE
OF THE
DRAINAGE COMMISSION
PROVINCE OF BUENOS AIRES

(3 170 gal.), and then conveyed in a pipe line, parallel to the bridge, with outlets at intervals of 14 m. (46 ft.).

The foundations of the pylons are of reinforced concrete, and in these the materials and their proportions are the same as in the plain concrete. In all the foundation concrete, granite rubble was embedded in order to economize in crusher stone, because, owing to a variation in the original design, a large surplus of this material was available.

Each pier supporting the central span rests on a reinforced concrete slab, and this rests on two reinforced concrete cylinders which were sunk to hard clay 9 m. (29½ ft.) below ground level. The internal diameter of each cylinder, at the steel cutting edge, is 3 m. (9 ft. 10 in.), decreasing to 2 m. (6 ft. 6¾ in.) at the top. The lower section of each cylinder was built to a height of 3 m. (9 ft. 10 in.) and at once sunk to the ground-water level, then the upper section was built and the sinking completed. The several steps in the work were as follows:

1.—Excavation, to ground-water level, of area of slab;
2.—Construction of lower section of Cylinder I;
3.—Construction of lower section of Cylinder II; (the wooden framework over the excavation for handling buckets, etc., was erected while the cylinders were being built);
4.—Sinking Cylinder I and then Cylinder II to sand (2 m. below ground-water level), using picks, and shoveling the earth to the platform and to cars, a hand-diaphragm pump being used for unwatering;
5.—Construction of upper sections of Cylinders I and II;
6.—Sinking Cylinder I below the 2-m. level, using two buckets suspended from a pulley raised alternately by a hand-winch, the buckets dumping directly into cars, the unwatering being done with a 6-in. centrifugal pump with a steam motor.

Two men below kept the buckets in continual operation, and a crew of three alternated at the winch and dumped the buckets into the car. In ordinary hard but sandy soil the progress was about 1 m. of depth per day. There were no difficulties until, at a depth of 4 m. (13 ft.) below the ground-water level, quicksand was encountered. At the end of each pumping period, after a little excavation, there

was an inrush of fine sand which flowed like mud and filled the excavation at least 1 m. in depth. This material choked the flexible suction hose of the pump and caused delay. The constant inflow of sand began to undermine the upper crust and great fissures radiating from the cylinder excavation began to appear. Progress was stopped, although only 1 m. of fine sand separated the cutting edge from the hard clay; and, though the writer himself entered the water, it was difficult to restore the "morale" of the men.

The remedy for the trouble was simple. An inner cylinder of galvanized-iron sheeting was driven around wooden bracing rings, each sheet being driven into the hard clay. The inner cylinder was well cleaned, the pump suction pipe was hoisted, and the lower 2 m. (6½ ft.) of the cylinder concreted under water, using a wooden tremie to place the concrete, the tremie being raised by the hand-winch when necessary. The upper section was packed with sand and capped with concrete.

A reinforced concrete platform resting on the two cylinders completed the foundation.

Abutments and Piers.—The abutments and piers are of brick masonry, with 1:3 mortar. In part of the work washed stone crusher dust was used instead of sand, and with satisfactory results. In the central piers about 20% of the volume was rubble masonry filler owing to the surplus of rubble already mentioned.

Pylons.—The pylons are of 1:2:4 reinforced concrete, using 1 to 2-in. stone. They were built simultaneously with the piers, and all were at least 1 month old before the forms for the superstructure were erected. The columns were built in one continuous operation, allowing time for thorough settling of the wet concrete by working on two or more alternately, pouring sections 1 m. (3¼ ft.) high at a time.

Superstructure.

Forms for Lateral Spans.—The forms were designed to obtain the greatest recover value for use elsewhere. Two complete sets (each for one 28-m. (91.86-ft.) span) were used in rotation with three sets of uprights. This arrangement allowed for continuous work by the carpenters.

Movement of Materials.—Sand and stone were measured in the Decauville cars at the stock piles and transported to the central part

FIG. 1.—PYLON COLUMNS AND BRICKWORK PIERS AND ABUTMENTS, PICAZA BRIDGE.—SECOND MONTH.

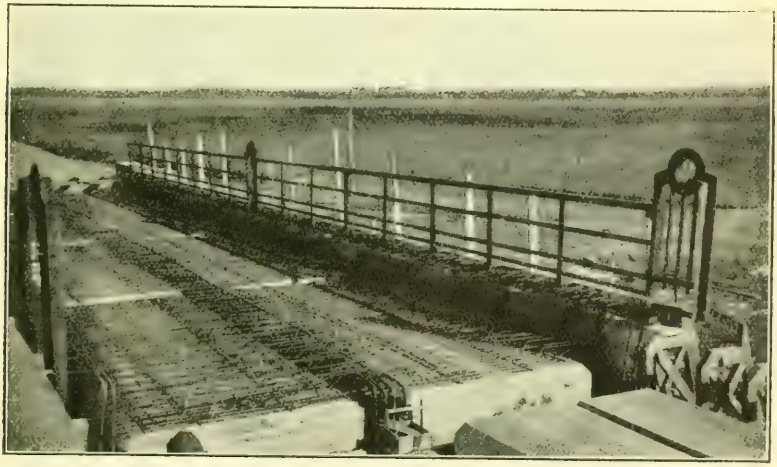


FIG. 2.—FORMS AND REINFORCEMENT OF HALF OF ONE LATERAL SPAN BEFORE ADJUSTING REINFORCEMENT.

of the bridge where two low-level mixing platforms received the stone after it had passed over an inclined slatted platform on which it was washed thoroughly with a hose. The sand was discharged directly to the platforms. Arrangements were made to permit the discharge of sand or stone to either platform, and the stone-washing crib or inclined platform was divided into four compartments, each having a capacity of 1 cu. m. (35.3 cu. ft.) so that washing could be carried on always two batches ahead of the mixing. No difficulties were experienced at any time with this arrangement.

Cement and sand, in the proportions of 1:2, were mixed by hand, dry, turning the mixture three times, until it was of uniform color; then two parts of washed stone were added, and the concrete was mixed with plenty of water until the mass was of uniform jelly-like consistency.

A 50-ft. steam derrick with a bucket containing $\frac{1}{3}$ cu. m. (11.7 cu. ft.) received the concrete directly from the mixing platform and hoisted it to an elevated distribution trestle parallel to the bridge.

The concrete was dumped from the bucket of the derrick into a Decauville car on the trestle, and the car ran by gravity to the point of discharge, where it was placed by a movable dumping box, 2 by 2 by 0.30 m. (3.28 by 3.28 by 1 ft.), with adjustable, steel chute (a 0.30-m. smokestack sawed in two). There was no trouble of any kind with this arrangement, and concrete was in place within half an hour of mixing. Two men handled the car and derrick bucket, and dumped the concrete into the dumping box; two men below tamped the concrete, and controlled the discharge of the chute; and a fifth man acted as general emergency man, a very important position, as the conditions of discharge varied continually. It may be noted that all concrete was mixed very wet, and all forms were drenched before they were filled, although, of course, all surplus water was drained off.

One elevated trestle was used for one-half of the bridge, and after concreting, it was taken down and re-erected for the other half. The truck was about 2 m. (6.56 ft.) above the bridge floor and about 1 m. (3.28 ft.) from the bridge railing over which it was necessary to chute the concrete. The concrete was in the proportions, 1:2:2. The sand was a very good, coarse, river sand from Uruguay. Fine granite chippings were used for the lower parts of the beams, where

the complicated reinforcements demanded a very fine aggregate, and a 1 to 2-in. stone in the remainder of the concrete.

With one gang of 10 men working alternately on the two mixing platforms, it took 3 days to concrete one 28-m. (91.86-ft.) span, the work being executed in the following order:

1a.—All reinforcement was securely placed, tied, and braced.

b.—All forms were washed and well drenched.

c.—All concrete adhesion surfaces (pylons) were picked and grouted with neat cement.

d.—All surfaces of contact over masonry pillars were rubbed with clay to prevent adhesion.

e.—Sliding bearing plates for beams to rest on pillars were placed and greased face to face.

2.—*First Day.*—Longitudinal beams were concreted from the pylon outward, on both sides to the center of the 14-m. (45.93-ft.) span, leaving a vertical joint near the center of each beam.

3.—*Second Day.*—The vertical joints were washed, brushed lightly, and grouted with neat cement. The beams were finished to the end of the span.

4.—*Third Day.*—After riveting the railing and adjusting the horizontal hand-rail to exact position (it is embedded in the concrete of the sidewalk), the slab was concreted, beginning at one end and finishing at the other.

Great care was taken throughout to ensure bond by washing and grouting, and every precaution was taken against the displacement of reinforcement or of concrete already set.

Central Span.—Entirely new forms were required for the central span. The steel reinforcing arch was utilized as much as possible to support the forms for the arch, and the transverse beams and slabs were supported from below.

The work was executed in the following order:

1.—The steel reinforcing arch was erected and bolted. This was done with the customary falsework, which was used subsequently to support the floor forms. The arches were braced by wooden struts and tied with wire while being adjusted and bolted.

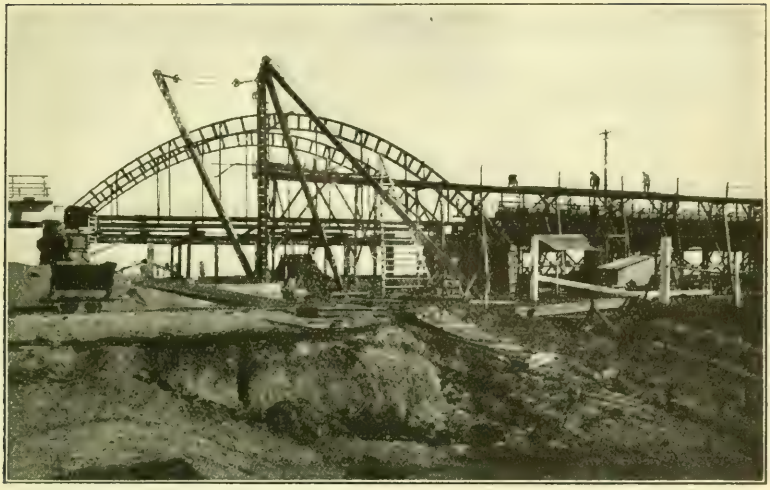


FIG. 3.—CENTRAL SPAN, MELAN REINFORCING ARCH. ELEVATED TRACK TRESTLE FOR END SPANS, PICAZA BRIDGE.

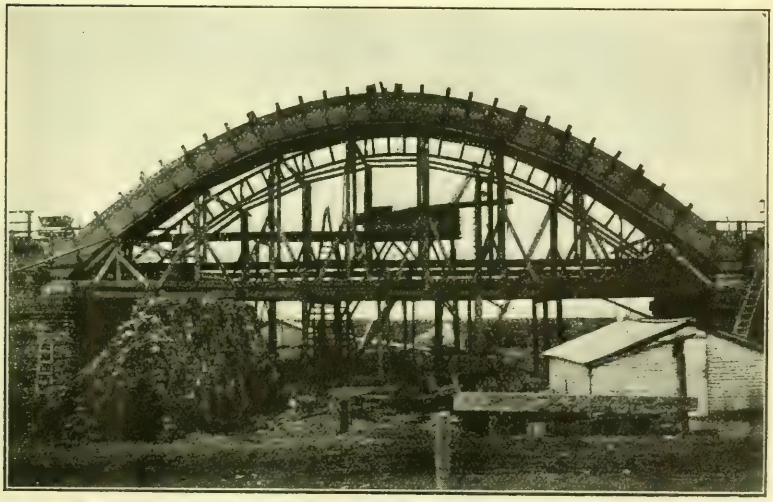


FIG. 4.—CENTRAL SPAN, PICAZA BRIDGE. FORMS FOR ARCH.

2.—The forms for the concrete arch and suspended floor system were built, using material from the old forms where possible.

3.—The arches were concreted with 1:2:1½ concrete, using fine stone. One arch was completed in one day, beginning at the haunches and working toward the crown, with alternate pourings, so as to distribute the weight symmetrically, thus no construction joints were necessary.

4.—The floor-beams, slabs, and sidewalks were concreted in the order named, and the pavement and sidewalk finished as that of the rest of the bridge.

5.—One month after concreting the arches, in order to allow for subsequent deflections, and with all dead load in place (paving block, sand, etc.), the hangers or suspension members were concreted, and afterward the longitudinal tension members.

After concreting the lateral span of the first half of the bridge, the masonry on that side was continued upward and, at the same time, granite curbing and compressed asphalt tile sidewalk were placed.

Pavement.—The concrete floor was water-proofed with a 1-cm. (0.39-in.) coating of a coal-tar sand mixture placed hot and ironed. All expansion joints were filled with asphalt. Surface and sub-surface drains were placed, and the floor of the bridge was paved with Swedish granite block paving sets, consisting of 0.10-m. (3.9-in.) blocks, placed so as to form a mosaic fan-shaped design. These blocks were laid on a 7-cm. (2¾-in.) cushion of sand, and the joints were filled with coal-tar after the pavement had been consolidated thoroughly.

The abutments were paved with ordinary 15-cm. (5.9-in.) block paving resting on a concrete raft slab, to allow for subsequent settling. Although the fill on which the pavement rests was consolidated thoroughly by watering and tamping, it had settled 5 cm. (2 in.) one year after it was laid. It had been given a super-elevation of 7 cm. (2¾ in.).

Concluding Masonry, Ornamental Designs, etc.—After concreting the central span, the masonry of the central pillars was finished and the ornamental designs at the crowns of the arches and at the haunches were added by grouting behind plaster of Paris negatives held in place by bracing and wiring. Granite mosaic work was used to fill the decorative triangular panels above the haunches of the arches,

and it may be of interest to note that this mosaic gives a very effective and beautiful contrast to the concrete.

The ornamental brackets on the side spans were cast in Buenos Aires on a foundation of expanded metal. They are hollow, and very carefully hammer-dressed to resemble cut stone. They were tied in place to the sidewalk reinforcement through tubes, purposely left in the concrete, which were afterward grouted from above and sealed.

Surface Finish.—All masonry above the cornice level was plastered with 1:2 sand mortar, and, after hardening, was hammer-dressed to give the appearance of cut stone. The cornices were plastered with the same mortar but combed with a fine steel comb, giving an imitation of cut stone. This method is used effectively in the Argentine on nearly all buildings of architectural merit.

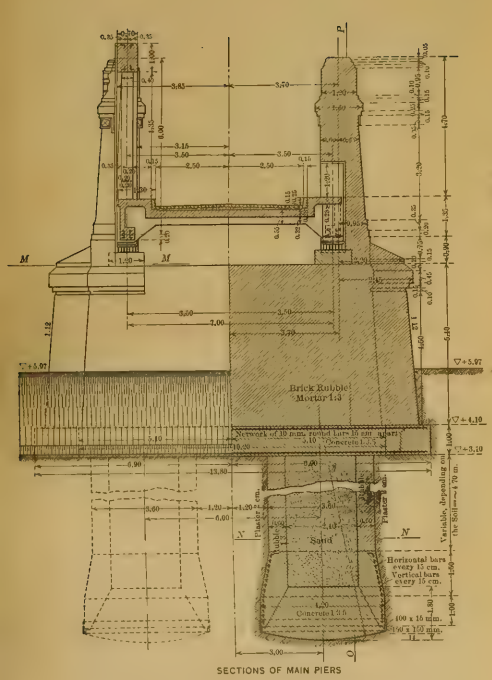
Below the cornice level a rough granolithic surface was secured by first making longitudinal grooves with cement mortar and, when these were hard, plastering between the grooves with granite chips, and cleaning the surface subsequently. This produces a granite surface which reflects the light and is broken by smooth horizontal lines—a combination which contrasts well with the smooth pylon columns.

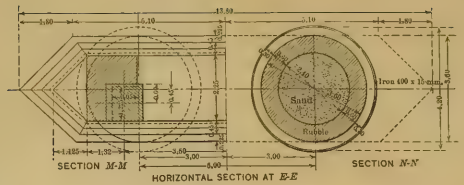
The exposed sides of the side spans were left rough and scrubbed with wire brushes; then a coarse sand mortar was rubbed on the rough surface, using small grindstones for that purpose; two coats were applied, the second on the first before it had set. In this way a hard impermeable rough surface was obtained, which, at a distance of 3 m., can hardly be distinguished from the hammer-dressed portions. It may be pertinent to call attention to the fact that a surface of uniform color and texture was desirable because the necessary use of different cements on different spans had produced a variety of shades, not at all pleasing, and this finish corrected this defect.

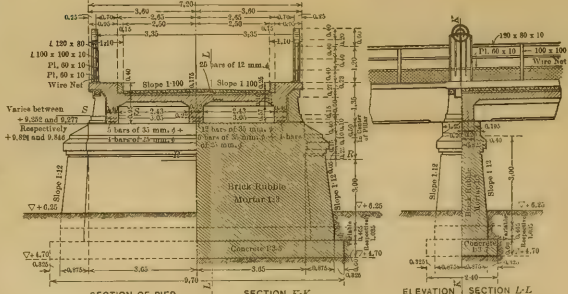
The upper and lower surfaces of the arches were finished smooth. The hangers and tension members were plastered on the picked concrete, in order to obtain the curves of the design. The sides of the arches were well picked and then plastered with fine granite chippings, and wire-brushed 12 hours afterward to remove the cement coating. A fine and very satisfactory granolithic surface was obtained. The under surfaces were simply washed with cement to produce a uniform tone.

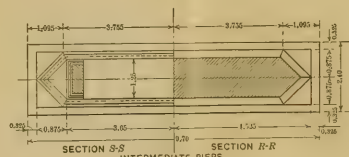
FIG. 5.—GENERAL VIEW OF PICAZA BRIDGE, AFTER OPENING TO TRAFFIC.

FIG. 6.—GENERAL VIEW OF PICAZA BRIDGE.






Slope 1:10
25 bars of 12 mm. ϕ
Varies between + 9.395 and 9.277 Respectively + 6.898 and 9.046
5 bars of 35 mm. ϕ + 4 bars of 25 mm. ϕ
12 bars of 30 mm. ϕ + 4 bars of 25 mm. ϕ
Brick Rubble Mortar 1:3
Concrete 1:2:4
Wire Net
SECTION OF PIER
SECTION K-K
ELEVATION
SECTION L-L



PICAZA BRIDGE
DETAILS OF
MAIN AND INTERMEDIATE
PIERS

In all cases the objects have been to obtain a uniform tone, to break up the light, and, at the same time, to avoid monotony in the appearance of the different classes of surface. It may be said that these objects have been achieved, although, as the granite used is of exactly the same color as the cement, far greater and more pleasing contrasts could have been obtained with an aggregate of a different color.

The appearance of the bridge is pleasing, and it is of interest to note that although a near view gives the idea of massiveness, at a distance of 1 km. (3 280 ft.), the impression conveyed is that of extreme lightness.

General Observations.—After a year of sun, frost, and floods, it has been noticed that there are no structural cracks in the work. A few have appeared in the mortar plastering, but always near or at expansion joints or bearings of the concrete beams, and these, in all cases, have been due to the plastering of the brickwork not having been isolated completely (by cutting) from the movable beam surfaces.

The effect of variations in temperature on the length of the beams is very noticeable. On a cold night a movement of 1 cm. (0.39 in.) is easily seen on the lateral spans, and the central span contracts 2 cm. (0.79 in.). The range of temperature is from 0° to 35° cent., but the most sudden changes are in the early spring and late fall when freezing temperature at daybreak is followed by high temperature at noon, or within 5 hours.

Progress According to Schedule.—A schedule of work was made and adhered to as closely as possible, but there were many causes for delay. For instance, there was a compulsory slow-down of 2 months caused by a general strike of the railroad locomotive drivers; materials were delayed in arrival, the rate of work was lessened, and, in fact, all work was affected. Another cause of delay was in the sinking of the cylinders, which was not as rapid as expected, owing to the difficulties mentioned. This was an important feature because the completion of the central piers controlled the time of finishing the spans resting on them. There were also minor delays caused by modifications in design, lack of paving materials (which were imported from Sweden and were late in arrival), and for 3 months at the beginning of the work there were not enough competent carpenters for form work. The work began in January, 1912, and

traffic was passing over the bridge in December of the same year, although all the surface treatment had not then been completed.

Costs.

Table 2 shows the scale of wages paid. The working day varied from 7 hours in winter to 10 hours in summer. Italian, Spanish, Greek, and Bulgarian laborers were employed.

TABLE 2.—Scale of Wages Paid.

Duty.	Wages or Salary.	Equivalent American Currency.
General foreman	$220.00 and bonus, average $280.00	$123.00
Concrete foreman	125.00 " " " 150.00	66.00
Concrete laborers	3.20 " " " 4.50	1.98
Cylinder laborers	4.00 " " " 5.50	2.42
Carpenters	6 to $7 " " " 8.00	3.52
Masons	5 to 7 " " " 8.00	3.52
Time and storekeeper	120.00 120.00	52.80

Tables 3, 4, 5, and 6, showing the costs, are self-explanatory. Owing to the fact that the costs of materials, transportation, and overhead charges not incurred on the work itself, were not kept by the writer, he regrets that he cannot detail them as he has the labor costs, but the totals are correct and he trusts that they may give an idea of the cost of the work under given conditions in a distant country. To an American engineer, the proportion of money spent on purely esthetic grounds in a prairie region may seem to be too great, but such seems to be the custom of the country, and perhaps, eventually, on broad grounds, it may be regarded as proper for permanent structures of this type.

TABLE 3.—Costs of Labor and Materials on Picaza Bridge.

Class of work.	Cost.	Units.	Cost per Unit.		Observations.
			Pesos.	U. S. gold dollars.	
Excavation, foundations	791.16	1 023.3 m.	0.77	0.33	Average depth, 1 m.; average haul, 150 m.
Excavation, sinking cylinders	2 015.08	283 m.	7.12	3.03	To 9 m. below surface. Haul, 200 m. includes pumping.
Cylinders, sand filling	71.71	50 m.	1.43	0.61	Including transport and loading at 200 m.
Concrete:—1 : 3 : 5					
Foundations	984.03	301 m.	3.27	1.39	Mix and place, including some reinforcement.
Cylinders	886.17	174 m.	5.09	2.16	Including forms, reinforcement and plastering.

TABLE 3.—(*Continued.*)

Class of work.	Cost.	Units.	Cost per Unit.		Observations.
			Pesos.	U. S. gold dollars.	
Reinforced Concrete :— 1 : 2 : 2 and 1 : 2 : 1½	.....	.....	.....	.....	1 : 2 : 1 ½ only in arches and hangers, 48.3 m.
Reinforcement, prepare.	670.37	110 tons.	6.09	2.59	Bending and transporting from shop to place.
" place....	980.75	110 tons.	8.91	3.79	Place and tie.
Forms, erect and take down.....	5 751.12	732 cu. m.	7.86	3.34	Two sets used three times for lateral spans, one set for central span, cost 50% more.
Concrete, mix and place.	2 964.71	732 cu. m.	4.05	1.72	Central span = 125.3 m.
Masonry : Brickwork, 1 : 3 cement mortar.....	4 545.19	1 009.3 m.	4.50	1.91	250 tons of rubble utilized in masonry, including piers, abutments, and ornamental pillars.
Plastering on Brickwork : Imitation granite.....	1 349.96	529.2 m.	2.55	1.08	Granite chippings brushed, broken horizontally by smooth channeling.
Imitation fine cut stone...	2 180.50	623 m.	3.50	1.49	Plastered 1 : 2 fine sand and combed with fine comb.
Rustic.....	191.10	273 m.	0.70	0.30	Rough cast plastering.
Finishing Concrete Surfaces: Plastered and rubbed.....	986.64	793 m.	1.24	0.53	Plastered and rubbed with grind-stone, 2 coats.
Fine plastering.....	1 627.50	465 m.	3.50	1.49	Plastered 1 : 2 fine sand and combed with fine comb.
Imitation granite.....	267.00	89 m.	3.00	1.27	Granite chippings brushed, broken horizontally by smooth channeling.
Cement wash.....	338.40	1 692 m.	0.20	0.09	Applied with brush, 2 coats.
Hammer-dressing Mortar Surfaces.....	584.48	244 m.	2.39	1.02	On plaster 80%; on narrow concrete bands, 20 per cent.
Modifications : Tie-wall pylons.....	481.75	161 m.	29.92	12.72	Between columns, including picking sides of columns and reinforcement.
Transverse arches.....	119.63	3 m.	39.87	16.94	Central span wind bracing, including forms, reinforcement and plastering.
Ornamental Castings : Central span.....	269.40	12 No.	22.45	9.54	Place negatives, grout with 1 : 2 mortar, and finish surfaces.
Laterals-brackets.....	335.76	84 No.	3.99	1.70	Place, grout, and plaster bevels, etc.
Steel Hand-railing : Rectify.....	212.00	380.1 m.	0.56	0.24	Made in Buenos Aires, received bent.
Place and rivet.....	604.76	380.1 m.	1.59	0.68	Including bracing while concreting sidewalks.
Paint, 3 coats.....	484.79	271 tons.	1.79	0.76	
Pavement : Water-proofing concrete..	396.00	990.2 m.	0.40	0.17	Coal-tar and sand applied hot and ironed.
Surface and sub-surface drains.....	76.42	36	2.12	0.90	16 surface cast-iron inlets ; 16 sub-surface 2½-in. pipes, and 4 inlets, cast iron and brick, on abutment areas.

TABLE 3.—(*Continued.*)

Class of work.	Cost.	Units.	Cost per Unit.		Observations.
			Pesos.	U. S. gold dollars.	
Asphalt expansion joints.	85.43	6	14.24	6.05	Each about 3.50 sq. in. and 0.05 in. thick (between spans).
Swedish block paving.....	2 080.00	1 040.2 m.	2.00	0.85	10-cm. cubes placed in fan-shaped mosaic and joints tarred.
Granite curbing.....	608.33	426.1 m.	1.43	0.61	$286.45 cutting to required section.
Concrete base (abutments).....	72.49	14.7	4.93	2.10	To allow uniform settlement of pavement.
Sidewalk (asphalt tiles)..	273.26	278.2 m.	0.98	0.42	($3 591.93 pavement.)
Steel Reinforcing Arch :					
Central span, including bearings.....	1 186.52	24 tons.	49.44	21.01	Including falsework and adjusting.
Structural steel in expansion joints.....	81.95	.....	.....	.....	To reinforce brickwork.
Wash and clean broken stone.....	523.06	1 000 tons.	0.52	0.22	Very dirty from being deposited 2 years in exposed place.
Handling materials on work.....	1 238.90	5 540 tons.	0.22	0.09	From stone piles, including loading and transporting 150 m.
Unloading from carts.....	4 561.60	6 900	0.66	0.28	Materials and plant 60% done 1 year before work began, and without supervision.
Back-fill and tamping.....	307.80	.....	.....	.....	In 12-in. layers, and wet.
Concrete trestle — erect and take down.....	1 399.17	Twice.	699.58	297.32	Trestle 100 m. long, average height 6 m., erected and taken down twice.
Repairs to plant and equipment.....	893.41	.....	.....	.....	Tools, derrick, etc.
Camp :					
Installation.....	1 338.05	.....	.....	.....	Including $735 prior to commencing work.
Removal.....	436.66	.....	.....	.....	
Drainage.....	700.70	.....	.....	.....	
Deviation of highway.....	709.55	.....	.....	.....	To maintain traffic while building bridge.
Ramps and Level Crossings:					
Formation of.....	6 761.16	11 542.6	0.585	0.25	10 293.3 m. ramps, level crossings 1 249.3 m.
Sodding.....	342.11	.....	.....	.....	
Fencing ramps.....	386.13	.....	.....	.....	
Testing bridge.....	154.88	.....	.....	.....	
Total labor.....	54 307.54	.....	.....	.....	
Administration :					
On work.....	9 262.69	.....	.....	.....	Cook, inspector, etc. Of this, only $5 678 belongs to construction period, the $3 131 was due to delay in beginning work.
General overhead charges from Dolores office.....	8 787.03				
Total administration..	18 049.72				

TABLE 4.—Approximate Cost of Materials, Picaza Bridge.

Materials.	Units.	Unit price.	Cost.	Observations.
Cement	672 tons.	$61.24	$41 153	As these costs of material are posted monthly by the Dolores office, the unit prices given are approximate averages. All freight handling charges at Buenos Aires and Dolores are included.
Sand	1 680 "	4.90	8 232	
Broken stone and screenings	1 914 "	6.10	11 675	
Rubble stone	250 "	4.20	1 050	
Steel reinforcing bars, round	110 "	118.00	12 980	
Steel, structural in arch	24 "	407.83	9 788	
Steel, ornamental railing	27 "	618.52	16 700	
Brick	360 000	23.00	8 280	
Swedish granite paving sets	1 040.2 m.	7.00	7 280	
Curbing	4 461 m.	7.00	3 122	
Asphalt tiles for sidewalk	4 m.	400.00	1 600	
Ornamental brackets	83	8.29	688	
Ornaments (negatives)	4	40.00	160	
Wire, bearing plates, etc	.....	.....	4 500	
Carting	6 900 tons.	2.00	13 800	
Total	.....	.....	$141 008	

TABLE 5.—Résumé of Cost of Picaza Bridge.

Labor		$54 307
Materials (transportation, etc., on work)		141 008
Plant: Depreciation	$5 000	
Wood, coal, etc., consumed	6 000	11 000
Administration and supervision		18 049
Grand total		$224 364

TABLE 6.—Costs of Picaza Bridge per Unit of Length and Surface.

	Length, Over All.		Paved Area, Over All.	
	Including abutments.	Excluding abutments.	Including abutments.	Excluding abutments.
Quantities	210 lin. m.	197.80 lin. m.	1 494 sq. m.	1 424 sq. m.
Pesos per linear meter	1 068.40	1 134.30	.....	.....
Pesos per linear foot	325.63	345.72	.....	.....
U. S. gold dollars per linear foot	$138.39	$146.93	.....	.....
Pesos per square meter	.....	.....	150.10	157.50
Pesos per square foot	.....	.....	13.95	14.64
U. S. gold dollars per square foot	.....	.....	$5.93	$6.22

Total volume of masonry, including concrete = 2 215.3 cu. m. = 2 899 cu. yd.
Area, over all (excluding abutments)..... = 1 424 sq. m. = 15 322 sq. ft.
Equivalent to 0.19 cu. yd. per sq. ft.
$1.00 U. S. currency = 2.354 pesos.
1 peso = $0.425 U. S. currency.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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PEARL HARBOR DRY DOCK.

By H. R. Stanford,* M. Am. Soc. C. E.

To be Presented September 1st, 1915.

Synopsis.

The object of this paper is to inform those interested in engineering matters regarding the peculiar difficulties and problems which the construction of the Pearl Harbor Dry Dock has involved.

The paper contains a topographical and geological description of the location which was chosen for the dock, a review of the Congressional authorizations and appropriations for the work, a brief description of the plans and methods of construction originally adopted, an outline of the changes which were made in dimensions and details as the work proceeded, and a description of the field troubles which were experienced and of the experiments and tests conducted to determine the best means for their correction.

The paper concludes with a description of the plans and methods which were finally adopted and under which the work is now proceeding.

A paper on the Pearl Harbor Dry Dock at this time may on first thought be considered premature, inasmuch as little progress has been made toward actual completion, but a description of the work from its inception to date seems to be justified, in view of the numerous

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Corps of Civil Engineers, U. S. Navy; Chief of Bureau of Yards and Docks, Navy Department.

and peculiar construction difficulties which have been experienced, its military importance, its interest as an engineering project, and the recent resumption of operations on radically modified plans. It must be borne in mind, while reviewing the history of this project, that we now have the inestimable advantage of "hindsight" for our guidance, and that the problems involved should be weighed accordingly.

Location.

Pearl Harbor is in the southern part of the Island of Oahu, of the Hawaiian group, and is a body of water of irregular shape, formed by several lochs or arms of the sea. The depth of the water in the harbor generally varies from 35 to 50 ft., and these depths are well maintained to the shore line. The land surrounding the harbor is comparatively level, at an elevation of about 12 ft. above mean low tide. The tidal movements average about 1 ft. 2⅜ in. The channel connecting the harbor with the sea was naturally shallow and rather crooked, but, as a result of dredging operations completed in 1912, it now has a least depth of 35 ft. over an entrance width of 600 ft. and a minimum width of 500 ft. within the entrance, with curves eased to permit of the safe passage of battleships; the dredging operations required the removal of 4 645 000 cu. yd. of material, throughout a channel 5 miles long, at a cost of $3 296 000, and are of engineering interest because of the successful manner in which the work was conducted over the entrance bar, where there was a heavy sea swell at all times.

The area included in the Pearl Harbor Station was acquired in February, 1904, by the Bureau of Equipment of the United States Navy Department under its authority to purchase land required for the storage of coal.

The Act of Congress, approved May 13th, 1908, making appropriations for the naval service, includes an item reading:

"The Secretary of the Navy is hereby authorized and directed to establish a naval station at Pearl Harbor, Hawaii, on the site heretofore acquired for that purpose; and to erect thereat all the necessary machine shops, storehouses, coal sheds, and other necessary buildings, and to build thereat one graving dry dock capable of receiving the largest vessels of the Navy, at a cost not to exceed two million dollars for said dry dock."

The location of the naval station is especially favorable because of the proximity of natural deep water to the large areas of low-lying level land which are included within the boundaries of the property. The ground conditions are fairly good for the support of heavy concentrated loads, but sub-surface operations, which require the exclusion of ground-water, are costly because of the porous character of the soil. The railroad and highway facilities between Honolulu and the station are excellent, and the existence of the large and convenient commercial center is of inestimable value to the station as a base for labor and supplies.

The first general outline plan for the development of the Pearl Harbor Naval Station, following its Congressional authorization, was submitted in a report dated July 22d, 1908, by a Board of which Rear-Admiral Seaton Schroeder, U. S. N., was senior member. The location for the dock shown on this plan was slightly changed in its axial and transverse position, and the design was directed for a dock having a length of 1 140 ft. with an intermediate caisson to provide two chambers, 564 and 536 ft. long, respectively, in accordance with the recommendation of C. W. Parks, M. Am. Soc. C. E., Civil Engineer, U. S. N., and the Bureau of Yards and Docks, by authority of the Navy Department dated November 19th, 1908. The location for the dock was again changed by the Navy Department on March 19th, 1909, by moving the dock on its axis a distance of 200 ft. inshore, as a result of the great apprehension expressed by bidders regarding the dangers and difficulties which would attend the building and maintaining of a coffer-dam in the deep water in front of the dock entrance.

Geological.

The Island of Oahu is of volcanic origin, and consists of an elongated hilly or mountainous section, about 45 miles in length, lying in a southeast and northwest direction, with a surrounding fringe of low land formed by accretions of coral and volcanic origin. Pearl Harbor is on the south shore of the island, within the fringe formation, and is about 8 miles from Honolulu.

The geological conditions at the site of the dock are of very great importance in the consideration of the difficulties which have been experienced in the construction of the work, and therefore warrant a careful description. The surface soil, to depths exceeding those

reached by any part of the dock, is either volcanic or organic sediment. The volcanic sediment or tuff beds result from the intermixture of ashes and volcanic stones fallen from the air, with pebbles and sands made of hard lava washed in from the island valleys, the combination being, in some cases, re-arranged by water into beds of sand, or mud, or gravel. The material of organic origin consists of coral skeletons or shells ranging from hard reef, as originally deposited by coral animals, to broken and disintegrated material heaped together by the waves or assorted into gravels, sands, and muds, or intermixed with volcanic materials. There is no regular stratification of the material, and its lack of uniformity and its wide variation between the limits of hard and soft may be judged from the penetration records of various pairs of adjacent piles (piles in any one pair not more than 4 ft. apart) which were driven to refusal in the bottom of the dock excavation, as follows: 9 and 23 ft.; 5 and 26 ft.; 6 and 25 ft.; 3 and 24 ft.; 22 and 37 ft.; 11 and 35 ft.; 15 and 43 ft.; 5 and 32 ft.; 13 and 21 ft.; 11 and 22 ft.; 10 and 27 ft.; and 14 and 26 ft.

Described in detail, the soil comprises the following elements:

Rock Types:

Volcanic Tuff.—A hardened volcanic ash.

Volcanic Grit.—A hardened coarse volcanic sand.

Volcanic Agglomerate.—A hardened volcanic gravel made of angular fragments formed by explosion.

Coral Rock.—Material of an old reef still in place.

Soft Deposits:

Volcanic Mud.—A fine-grained aggregate of lava minerals formed by the wash of river or sea.

Volcanic Sand.—An aggregate of volcanic minerals, generally dark-colored, formed by wash of river or sea, and assorted to grains of fairly uniform size. The grains will be more or less rounded, according to the distance they have traveled and the amount of wear to which they have been subjected.

Volcanic Gravel.—Like volcanic sand, but coarser and of variable sizes. A true volcanic gravel containing pebbles of basaltic lava larger than the fragments in the Pearl Harbor tuffs, and occurring in their vicinity, is presumptive evidence of the

existence of a river channel from the hills, as it would be impossible for shore currents of the ocean to carry pebbles of ledge rock from any point on the shore of the island where such rock is exposed to the waves, along the coast and into the tortuous channels of Pearl Harbor without reducing them to sand or mud.

Coral Mud.—A fine-grained, white, yellow, or buff mud, made of crushed coral and shells.

Coral Sand.—Assorted grains of coral and shells of fairly uniform size.

Coral Gravel.—Washed fragments of coral and shells, more or less rounded, and more or less mixed with sand.

Mixed Deposits:

Tuff-Coral Mud.—A fine-grained aggregate of particles of tuff and of tuff minerals, mixed with particles of coral and shells.

Tuff-Coral Sand.—An aggregate of volcanic sand and coral sand mixed.

Tuff-Coral Gravels.—An aggregate of volcanic gravels and coral gravel mixed.

Basalt-Coral Gravel.—An aggregate of coral gravel and gravel made of partly rounded fragments of basaltic lava.

The softer classes of the foregoing materials become a mud or sand when soaked with water, but the solid coral and tuff rocks maintain their integrity. The volcanic deposits effervesce in dilute hydrochloric acid only in such proportion as they contain infiltrated carbonate vein material, whereas the corals, coral sand, and coral mud—which are composed almost wholly of carbonate of lime—effervesce very strongly.

An examination of the side slopes of the practically completed excavation for the dock, and of boring records, indicates a close resemblance between the underlying materials at the dock site and formations exposed in road cuts in the vicinity, except that the former are thinner-bedded and more uniformly horizontal, because of their greater distance from their source. Figs. 1 and 2 are photographs with notations which illustrate the principal formations. Borings for Artesian wells in the Pearl Harbor district reveal the great

thickness of these coral and volcanic deposits below sea level, thus clearly indicating a history of gradual submergence and explosive volcanic activities, with occasional uplifts of the deposits.

Design.

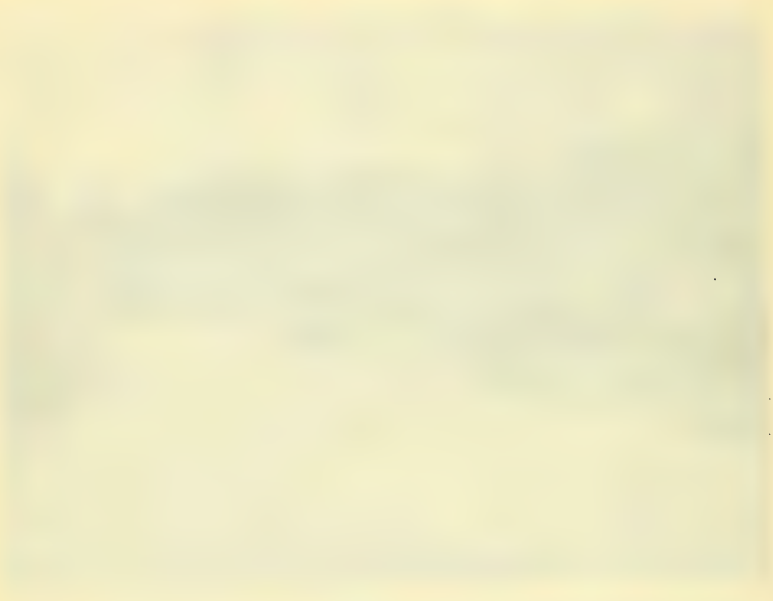
The usual modern dry dock of the graving type is essentially a masonry prism resting in an earth excavation. Inward earth and water pressure against the side-walls and upward hydrostatic pressure on the bottom are the principal forces which the prism must be designed to resist. The dry dock prism thus economically becomes a gravity arch structure, made of concrete, which depends on the weight of its material and arch action for stability. Water-tightness is incidentally obtained by the proper proportioning of the concrete ingredients. The main problem to be solved in the construction of a dock, therefore, depends on obtaining impervious monolithic and homogeneous concrete of permanent character, and having maximum unit weight combined with an ordinary degree of strength and hardness.

These comparatively simple requirements formed the basis of the design which was originally adopted for the Pearl Harbor dock. They assumed that concrete could be placed under the favorable dry and open conditions which usually obtain, and that earth friction on the sides of the prism, combined with the natural resistance of the underlying soil to infiltration, would warrant a reduction in the concrete in the prism to a quantity which would weigh somewhat less than the equivalent of a full upward hydrostatic pressure on the bottom.

As the work progressed, it became apparent that the usual open coffer-dam method of construction could not be followed, because of the porous and water-bearing condition of the soil. Arrangements were then made whereby foundation piling with heads buried in the bottom concrete was provided, and the contractor was permitted to place not to exceed the lower 8 ft. of bottom concrete by the under-water method. There was no provision, however, for any material enriching of the mix to offset losses and difficulties which the under-water method involved. The futility of this procedure was demonstrated by the defects in the 6 284 cu. yd. of concrete which had been placed to an average thickness of 6.52 ft. over a section of 180

FIG. 1.—ROADSIDE CUTTING, SHOWING MATERIALS FOUND IN EXCAVATION FOR PEARL HARBOR DRY DOCK.

FIG. 2.—ROADSIDE CUTTING, SHOWING MATERIALS FOUND IN EXCAVATION FOR PEARL HARBOR DRY DOCK.



lin. ft. at the inner end of the dock site, and which was partly exposed when the surrounding coffer-dam was unwatered to a depth of 37.3 ft.

The next modification in the work consisted in changing the ingredients in the under-water concrete, making the mix 1 part cement, 2 parts sand and 3½ parts broken stone, and in which one-third of the sand was silicious, from the mainland, and two-thirds were screenings from crushed lava rock. This richer mixture was placed to an average thickness of 6.75 ft. over a section of dock 240 ft. long adjacent to the inner section in which the lean concrete had been used. The under-water concrete in this section was placed in forms approximately 12 ft. square, and it was arranged that adjacent blocks should be finished with upper surfaces in planes separated by about 12 in., in order that an effective bond could be made between this lower concrete and that which was to be placed later when completing the floor. The collapse on February 17th, 1913, occurred in this second section, but, as all records agree that the concrete was of excellent quality, it may be asserted that the attempt to obtain satisfactory concrete in deep water by the tremie method was successful, and that the collapse was chargeable to other causes.

By this time it was recognized that local conditions demanded, not only unusual and expensive methods of construction, but also that the concrete should be exceptionally rich in quality with increased quantity, and that extensive metal reinforcements were necessary in the bottom to provide effectively for irregularities in the supporting power of the underlying soil. The recent change in contract recognizes all these needs; it provides a structure which is heavier than the water displaced; practically eliminates all concrete which must be placed under water; permits of thorough inspection; introduces substantial steelwork to relieve any probable tensile stresses in concrete; and provides for the independent settlement of seventeen practically equal portions of the dock before combining them into a single unit, thereby insuring the completed dock against settlement cracks and leakage.

Appropriations and Contracts.

Specific Congressional authorization is necessary before undertaking the construction of any public works improvement required for the Navy. The dry dock for Pearl Harbor was originally au-

thorized in an Act, approved May 13th, 1908, which fixed the limit of cost at $2 000 000. This limit was increased to $2 700 000 and then to $3 486 500 by Acts approved June 24th, 1910, and August 22d, 1912, in order that the dock could be made 1 000 ft. long and 110 ft. wide, to accord with the dimensions of the Panama Canal locks. The limit of cost was again increased by an Act, approved June 30th, 1914, and made $4 986 500, to provide for radical changes in plans to obtain a more substantial dock and reduce building hazards.

Contracts for naval public works always contain a provision whereby the Government reserves the right to make such changes in the contract, plans, and specifications as may be deemed necessary or advisable. The actual modification of a contract to cover a change of magnitude is accomplished with a document called a "Supplemental Agreement", which is acknowledged by the surety, accompanied with additional bond, and finally executed in the same manner as the original contract. Contracts and supplemental agreements for the Pearl Harbor Dry Dock in accordance with the Congressional authorizations previously cited, have been made as follows:

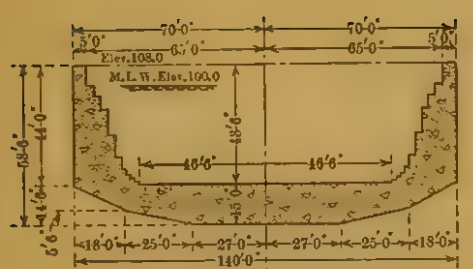
Original contract, dated July 22d, 1909, awarded to the San Francisco Bridge Company of San Francisco, Cal., provided for a dock to receive a vessel having dimensions not to exceed: length, 581 ft. 0 in.; beam, 105 ft. 4 in.; and draft, 31 ft. 8⅜ in., with clearance of 3 ft. 0 in. over the entrance sill, to cost $1 760 000.

Supplemental Agreement "A", dated June 27th, 1910, enlarged the dock to receive a vessel having dimensions not to exceed: length, 792 ft. 0 in., and beam, 113 ft. 4 in., and added $544 000 to the original contract cost.

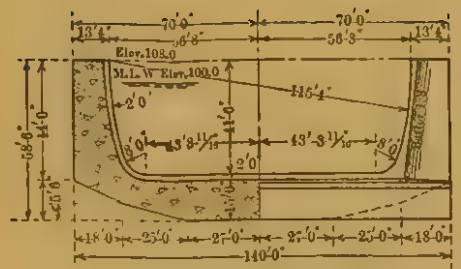
Supplemental Agreement "B", dated August 5th, 1911, provided foundation piles for the dock, with an increase in contract cost of $150 643.22.

Supplemental Agreement "C", dated January 2d, 1913, enlarged the dock to receive a vessel 1 000 ft. long, provided for enriching the bottom concrete, and added $711 842.50 to the contract cost.

Supplemental Agreement "D", dated November 19th, 1914, made radical changes in the details of the plans and specifications, without materially changing the dimensions of the dock, and added $1 261 350 to the contract cost. This addition, together with sums added on

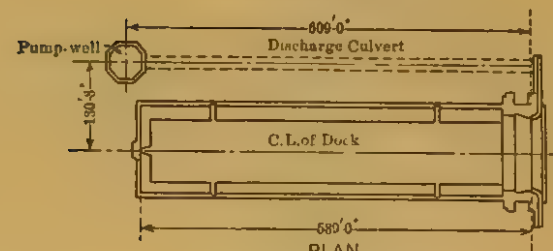


TYPICAL SECTION THROUGH DOCK BODY



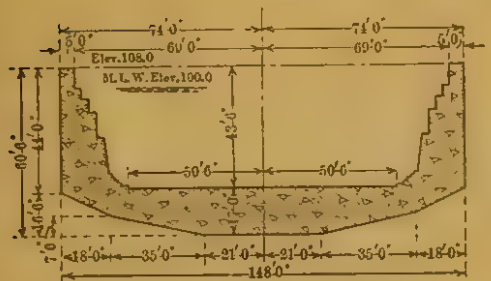
HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design covered by original Contract Dated July 22d, 1909

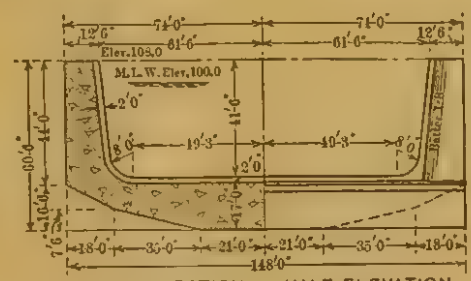


PLAN

Design covered by original Contract Dated July 22d, 1909

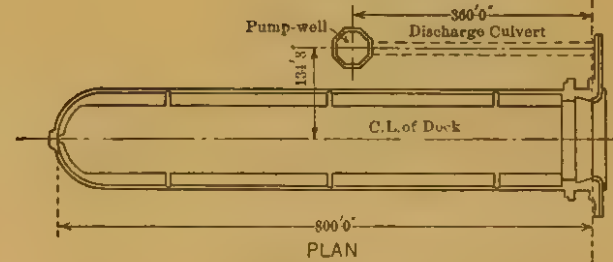


TYPICAL SECTION THROUGH DOCK BODY



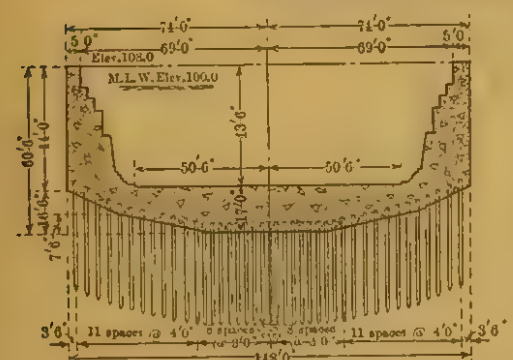
HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design after supplemental agreement Dated June 27th, 1910
Piling as shown below added by S.A. Dated, August 15th 1911

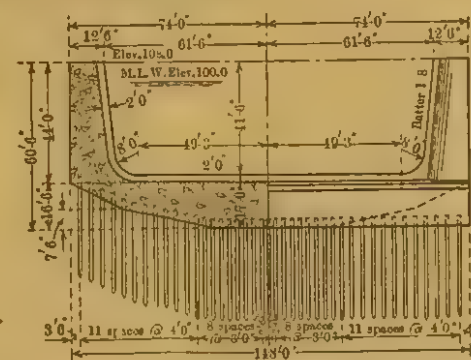


PLAN

Design after supplemental agreement Dated June 27th, 1910 and August 15th, 1911

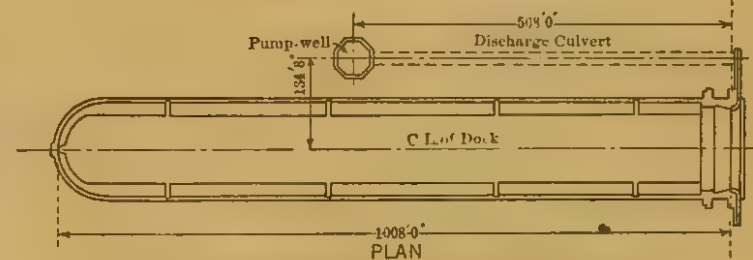


TYPICAL SECTION THROUGH DOCK BODY



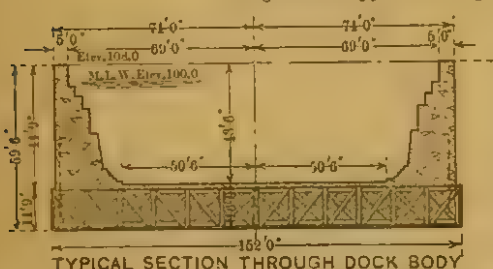
HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design after supplemental agreement Dated Jauuary 2nd, 1913

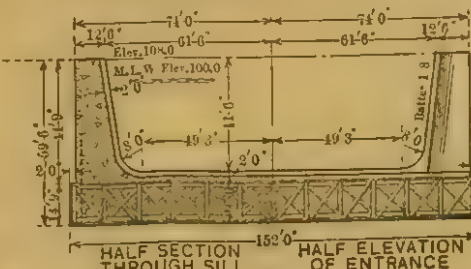


PLAN

Design after supplemental agreement Dated January 2nd, 1913

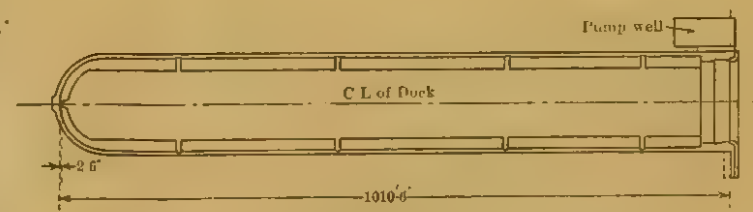


TYPICAL SECTION THROUGH DOCK BODY



HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design after supplemental agreement Dated November 19th, 1914



PLAN

Design after supplemental agreement Dated November 19th, 1914

DIAGRAMS SHOWING ENLARGEMENTS AND CHANGES IN THE DOCK

account of various minor changes, makes the total obligation under the contracts amount to $4 442 115.43.

Plate XXI shows the general outline dimensions of the dock as it has developed under the foregoing changes.

Supplemental Agreement "C" is considered of particular interest to the engineer because of the special provision which it contains to place full responsibility more definitely on the contractor for the completion of the dock, and also because of the relation which this condition created between the contractor and the Government's engineer in charge at the time of the collapse of the work on February 17th, 1913, and further because of its bearing on the negotiations preliminary to resuming work on radically changed plans subsequent to the collapse.

Prior to the execution of this supplemental agreement there had been a conference between the Chief of the Bureau and the contractors, during which the latter were informed that inasmuch as it was proposed to modify the contract to provide a dock of increased length, which would practically obligate the entire amount authorized for the construction of the dock, it would be necessary for the contractor to agree definitely to complete the work without any further increase in contract price, regardless of any mishaps or accidents which might subsequently be experienced. It was explained to the contractor that a primary reason for entering into contract for the construction of the dock was to insure the completion of the work within the amount authorized by Congress. In order to avoid interference with the operations of the contractors, which would tend to relieve them of their responsibility, it was arranged that the greatest possible latitude would be given in conducting the work; in other words, that it would be left largely with them to determine such details as the quantity of water which should be added in mixing concrete as affecting its consistency, the dimensions of the forms into which tremie concrete should be placed, the manipulation of the tremie, etc. In accordance with this understanding regarding responsibility, the following paragraph was included in the supplemental agreement forming a part of the contract:

"That it is specially understood and agreed that the party of the first part may elect any adequate methods or details of methods of construction to be employed in the construction of said dry dock and its appurtenances, and that the said party of the first part assumes full and complete responsibility for the adequacy of the materials specified and of the methods and details of methods of construction to

be employed and for the successful and satisfactory completion of the entire work in all its parts, and agrees to make good at its own expense any defect or inadequacy of the materials as herein or elsewhere specified and any defective or unsatisfactory work: *Provided,* That all materials and workmanship and the adequacy of all methods and details of methods of construction shall, as contemplated by the contract, be subject to the approval of the party of the second part. And it is further specially understood and agreed that all items of extras, damages, claims, time allowances, or other matters that have been or might be the subject of differences of opinion or dispute are merged in this agreement and satisfied and extinguished by the consideration herein stated and that no claim shall hereafter be presented or entertained on account of anything connected with the construction of said dry dock and appurtenances occurring or existing prior to the date of this agreement."

Having in mind this special provision of the contract and the circumstances which led to its adoption, let us consider the position of the officer in charge representing the Government during the several days immediately preceding the collapse of the work on February 17th, 1913. During this period the cribbing and sheet-piling protecting the section which was being unwatered had been showing increasing evidence of distress, up to the time of the collapse, when the bottom was almost exposed and in condition for the placing of concrete, each yard of which would have reduced the building hazard. It should also be noted that the conditions in this crib at this time were not very different from those which had existed several months before when Crib No. 1 was unwatered to practically the same depth without disastrous results.

The officer in charge, an experienced engineer, was fully cognizant of the extreme hazard of the situation prior to the collapse, but, knowing the responsibility on the contractor for the safe conduct and completion of the work, any interference on his part, based solely on his individual judgment, would probably have involved serious responsibility on the Government in defending his action as affecting the period required for the execution of the work, and also the contractors' expenditures. The great distance of the work from the Bureau and from other engineers with whom the officer in charge might have consulted, combined with the brief period during which the weakness of the coffer-dam was in evidence, together with knowledge of the contract provisions regarding the contractors' responsibility, resulted in placing the officer in charge in a very difficult position. Now that

the occurrence with all its details is a matter of history, it is very easy to conclude that some other course of procedure than that actually followed might have been better and proper; but, under the circumstances, it is doubtful if others would have done differently.

The negotiations between the Government and the contractors following the collapse of the work, were based on the claim of the contractors that the contract plans were insufficient to provide a safe and satisfactory dock; no claim was made on account of losses sustained. The contractors claimed, however, that it was the plans which were at fault, and that the Government was responsible for the plans; they further represented that any radical change in plans could not be considered as proper within the meaning of the paragraph on changes as contained in the contract, and that if such radical changes were desired it would be optional with the contractors as to whether they should agree to them. It was undoubtedly desirable that the contractors should proceed with the work in view of the valuable experience which they had acquired and in order to utilize the plant which had been assembled; the contractors on their part were anxious to proceed because of their business and professional standing. The Department of Justice, after a review of the contract, states, in an opinion given January 5th, 1914,

"Of many building contracts, I have never seen one that so persistently declared for full responsibility on the contractor during the course of construction."

In view of the foregoing conditions, the Bureau of Yards and Docks, which is charged with the design and construction of naval public works, felt constrained to withhold any proposal to adopt new methods or plans for the work until after the Secretary of the Navy should decide in favor of a stronger, more reliable, and more costly structure. This decision was reached by the Secretary early in April, 1914, after obtaining the advice of the late Alfred Noble, Past-President, Am. Soc. C. E., and various members of the Corps of Civil Engineers of the Navy; the general design for the dock which was subsequently adopted was then presented by the Bureau during the same month.

Field Operations.

Excavation.—Excavation for the dock was made with a dipper-dredge to a depth of from 35 to 38 ft. below low-tide level, and completed

to the required depth with a hydraulic dredge fitted with a rotary cutter. Dredging was begun in October, 1909, and practically completed in December, 1910. The material was loosened in advance of dredging by blasting, which required drilling 3-in. holes, from 6 to 10 ft. apart transversely, and somewhat farther apart longitudinally with the dock; about 50 lb. of 40% gelatine was used for each hole.

Coffer-Dam.—The open coffer-dam method of construction was adopted by the contractor when work was first started, and was followed up to the time that operations were temporarily suspended in February, 1913. The distance between the side-walls of the coffer-dam was about 148 ft., which was sufficient to include the full width section of the dock. The length of the coffer-dam was divided into sections by transverse bulkheads. The excavation required within a section of coffer-dam was completed in advance of building the coffer-dam. Interior cribwork was used for bracing the coffer-dam walls, as was done in constructing the dock at Mare Island, California, where the same contractors' superintendent was in charge. The walls of the coffer-dam were of 12-in. square timbers; two pieces of timber were first bolted together to form a unit, after which beveled-edged strips of 3-in. stock were secured to opposite edges of each unit, two strips on one edge and one strip on the opposite edge, the bevels and spacing of the strips being arranged so as to obtain a dove-tailed joint connecting adjacent units.

The cribwork comprised essentially a system of six horizontal tiers; each tier was of transverse and longitudinal timbers 12 ft. from center to center; a vertical timber standing on a concrete footing piece at the bottom of the excavation bound together the several tiers at each panel point of each bent; the upper tier was about 3 ft. above low-tide level, and the lower tiers were at intervals of 11.5, 10.1, 9.0, 8.0, and 8.0 ft., respectively; the first, second, and sixth tiers and the vertical members were of 12-in. square timber, with an 18 by 22-in. wale for the second tier; the third tier was of 14-in. square timber, with two 12 by 24-in. wales; the fifth tier was of 16-in. square timber with two 14 by 24-in. wales; the fourth and fifth tiers were braced together at each bent by intermediate transverse bracing to form Howe trusses; there was one line of diagonal bracing for each bent. Figs. 3 and 4 are photographs showing the coffer-dam and crib.

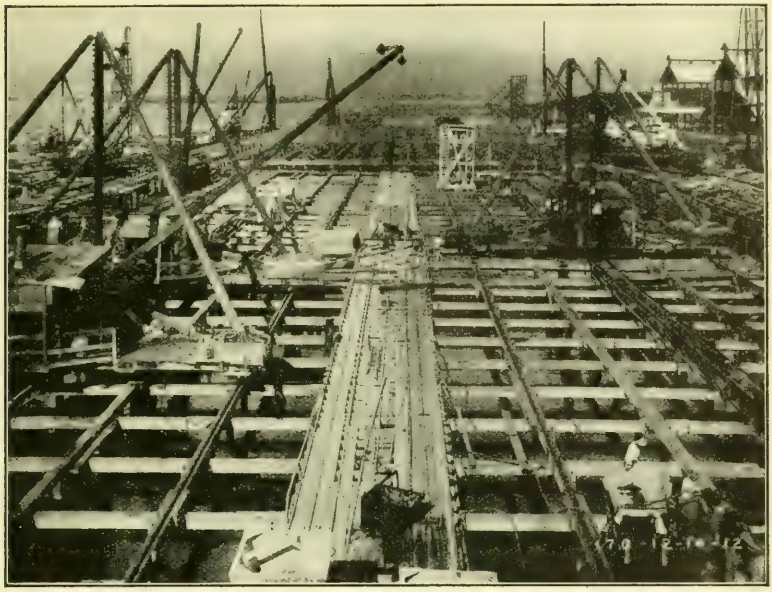


FIG. 3.—COFFER-DAM, PEARL HARBOR DRY DOCK.

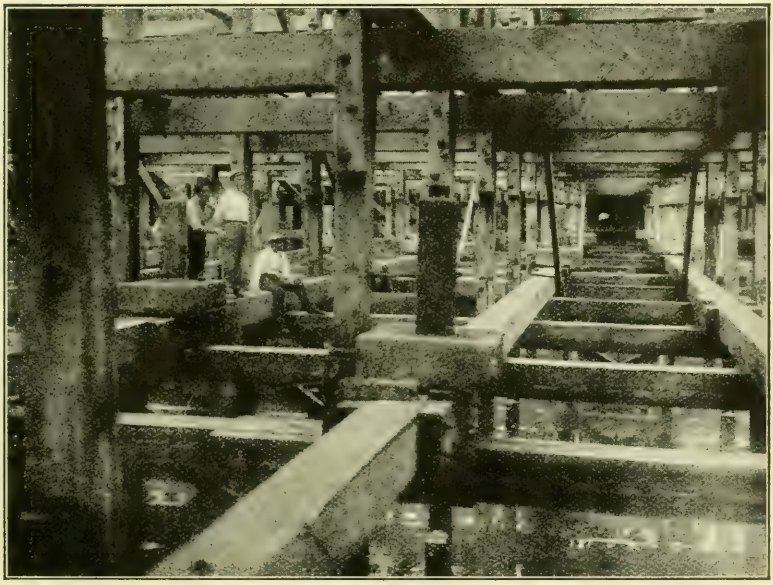


FIG. 4.—CRIBWORK IN COFFER-DAM, PEARL HARBOR DRY DOCK.

Pumping Crib I.—The first section of coffer-dam was completed during April, 1911, and enclosed 180 lin. ft. of the inboard end of the dock. Unwatering was begun on April 29th, 1911; the level of the water inside the coffer-dam was held from 2 to 3 ft. below sea level until May 3d while leaks in the sheet-piling were being located and stopped by divers. During the next 2 days the water was lowered to 16 ft. below sea level, when a slight rising was noted in the crib. On May 5th the water surface was lowered to 17.5 ft. where it was held until May 8th, when it was lowered to 21 ft. The cribwork gave evidence of serious distress when the depth of 21 ft. was reached, and the pressure was relieved by quickly reducing the head to 16 ft. The depth of 16 ft. was maintained until May 10th, when it was increased to 17 ft.; within a few hours after reaching this depth there was a sudden increase in the rate of leakage into the coffer-dam, until it exceeded the combined normal capacity of 18 500 gal. per min. of the three 15-in. and one 8-in. drainage pumps, and caused the suspension of pumping operations. When pumping was discontinued, the water level within the coffer-dam rapidly arose nearly to tide level, the water apparently entering principally through an opening which had developed in the sheet-piling. Pumping was resumed on May 11th, but, after 2 days, using all four pumps, it was found impossible to unwater below 16 ft. due to the excessive leakage into the coffer-dam, and further effort to unwater the section was therefore abandoned.

Very peculiar and alarming movements in the sheet-piling and cribbing had been noted during pumping operations; on May 5th, under a head of 17.5 ft., the cribbing in the southwest quarter of the coffer-dam had risen about 4 in. and the sheet-piling in the south half of the west side had risen $3\frac{1}{2}$ in.; on May 8th, with a head of 17.5 ft., the portions of crib and sheet-piling which had previously shown greatest movement had risen an additional $\frac{3}{4}$ in., and some rising had been noted over all other parts of the coffer-dam except in the northeast corner; after the head had been increased to 21 ft. on May 8th portions of crib and sheet-piling had come up $5\frac{1}{2}$ in. farther and were rising so rapidly, having reached a maximum of from $9\frac{1}{2}$ to $11\frac{1}{8}$ in., according to different observers, that the coffer-dam would have failed had the head not been quickly reduced; there was an immediate settlement of 1 in. in the sheet-piles and crib when the head was reduced to 16 ft.; the crib settled $6\frac{1}{2}$ in. and the sheet-piling $2\frac{1}{2}$ in. when the coffer-dam

was flooded on May 10th; the maximum permanent distortion noted on May 18th, after the coffer-dam had been flooded for 5 days following the suspension of pumping operations, was a rise of 6 in. in part of the sheet-piling and a settlement of $\frac{3}{4}$ in. in the crib. Observations on the test column (base area, 4 sq. ft., loaded with 16 tons, and located within the area of the coffer-dam to determine the bearing value of the soil) showed that the column rose slightly as the water in the coffer-dam was lowered, and settled when it was flooded.

Piling and Seal.—Experience acquired during the first attempt to unwater a section of the coffer-dam resulted in contract modifications, dated August 5th, 1911, which provided a piling foundation for the dock and permission to place concrete by the under-water method. The reason for the latter provision, and the limitation specified for its use, are given in the following quotation from the contract:

"For the purpose of forming a seal that will facilitate the unwatering of the excavation, concrete may be deposited under water. For this purpose tremies shall be used * * * . The depth of concrete so deposited shall not exceed 8 feet."

It was anticipated that the piles, which were to be 4 ft. from center to center longitudinally and about 3.5 ft. from center to center transversely, would have an average length of 39 ft., of which 4 ft. would be embedded in the concrete bottom of the dock, leaving 35 ft. for the depth of penetration. The piles would thus serve both as anchors and supports. The anchorage value of the piling naturally combined with the weight of the concrete seal to withstand the upward hydrostatic pressure on the concrete when the coffer-dam was pumped out; hence the greater the pile anchorage the less concrete required by the under-water method, and, as concrete placed through tremies was much more expensive than when deposited in the open, the less the cost to the contractors.

Contract payments on account of piling were based on the number of linear feet actually driven, and it was provided that the thickness of concrete in the bottom of the dock might be reduced 1 ft. to allow for the swelling of the ground which the driving of piles might produce. The reduction in the thickness of the concrete, as affecting the strength of the dock, was justified, inasmuch as the anchorage of the piling against hydrostatic uplift was more than equivalent to the weight of the concrete eliminated. The original plans for the dock

required that a 12-in. layer of rich concrete, 1 part cement to 4 parts stone, with enough sand to fill its voids, should form the outermost portion of the bottom and side-walls to serve as an impervious skin to exclude the percolation of ground-water into the interior mass of leaner 1:6½ concrete. The adoption of the under-water method for depositing concrete rendered it impossible to place these two grades as at first intended, and an average mix of about 1:3.4:6.19 was substituted, to be used alike for both tremie and dry concrete.

It was actually found that piles could not be driven to the depths anticipated, and the records show that the average penetration of the 1 966 piles driven in Section I, which includes 180 lin. ft. of dock at the inboard end, was 23.78 ft.; the 2 537 piles driven in Section II, which included 240 lin. ft. of dock adjoining Section I, have an average penetration of 20.53 ft.; the 2 652 piles driven in Section III have an average penetration of 28.49 ft.; and the 1 720 piles driven in Section IV have an average penetration of 28.20 ft. Pile driving was completed in Section I on September 11th, 1911; in Section II on December 2d, 1911; in Section III on June 13th, 1912; and in Section IV on January 11th, 1913. Piles are not yet driven for Section V. It was observed with surprise that, notwithstanding the close spacing and large number of piles driven, the surface of the ground between the piles showed little evidence of swelling or rising.

Placing concrete for the bottom of the dock and to form a seal in the bottom of the coffer-dam surrounding Section I was begun in October, 1911. It was soon found that the 1:3.4:6.19 mix was too lean to make satisfactory tremie concrete, and the use of a 1:3:5 mix was authorized on October 20th, 1911. Concreting through tremies in Section I was completed on January 4th, 1912; the 6 284 cu. yd. of concrete which were placed had an average thickness of 6.52 ft.

Pumping Crib I, Second Attempt.—The second attempt to unwater Section I of the dock was started on January 18th, 1912. The water surface was gradually lowered by four pumps having a nominal capacity of 18 500 gal. per min. A depth of 34.3 ft. was reached on January 26th, when it was noted that a portion of the cribbing of the coffer-dam near the west wall had risen 1 in. No further rise was noted in the coffer-dam 8½ hours later, when the head had been increased to 36.3 ft. The concrete in the bottom along the side-walls of the coffer-dam was exposed at this depth and found to be of generally poor quality,

and very poor in places. On January 28th the crib and sheet-piling near the center and west side of the coffer-dam had risen another inch, making a total of 2 in.; on January 28th, after the rise in the crib had increased to 2½ in., the head was reduced to 26 ft., where it was maintained until February 1st, while 160 tons of rock ballast were placed over the bottom at the places where the rise in the crib had been greatest. The head was then increased to 36.0 ft. without any further rise of the coffer-dam being observed. On February 2d the head was increased to 37.3 ft., and the removal of poor concrete and the substitution of richer concrete was started. Several leaks developed through the concrete seal on February 3d, and the pumps were stopped. The coffer-dam was completely flooded by leakage within 16 hours after pumping was stopped. On February 5th the crib had settled to within 1 in. of its original position on January 18th. On February 11th pumping was again started, and a head of 37.5 ft. had been obtained at 11.30 A. M. on February 13th when large leaks developed through the concrete, which resulted in pumping operations being stopped at 3.10 P. M., and the coffer-dam being allowed to fill. The defects in the concrete were apparently due to lack of experience in using the tremies under the conditions which prevailed and to the use of too lean a mixture, causing the impregnation of the concrete mass with sea water.

Concrete Changes.—The troubles experienced during the second attempt to unwater Section I led to contract changes dated January 2d, 1913, which required the removal of all the defective concrete previously placed in Section I, and provided for the use of a very much richer mixture for tremie-placed concrete. A considerable quantity of silicious sand from the California Coast was necessary for this new mixture. The increase in the dimensions of the dock covered by this contract change was made possible by the increase in the limit of cost which had shortly before been authorized by Congress.

The concrete seal made with the new and richer tremie mixture was completed in the coffer-dam surrounding Section II of the dock on January 20th, 1913. This section comprised the 240 lin. ft. of dock adjoining Section I, around which the coffer-dam had been constructed while concreting and pumping operations had been going on in Section I. The seal contained 8 555 cu. yd. of concrete having an average thickness of 6.75 ft. after making allowance for the volume

of embedded pile heads. In placing this seal, advantage was taken to provide a positive bond to connect the tremie concrete and the upper mass which was to be placed after unwatering; this bond was considered necessary to prevent the possible lifting of the upper mass of concrete in case leakage through the tremie concrete should produce internal disrupting pressures, and was to be accomplished by finishing the surfaces of the tremie concrete in adjacent forms at different levels; as the tremie concrete was poured into forms, each covering areas about 12 ft. square, this arrangement resulted in producing a surface somewhat like a checker board in which the surface of each 12-ft. square was approximately 12 in. above or below the surfaces of adjacent squares, thereby giving an effective dove-tailed bond with the superimposed concrete which was to be placed at a later date.

Pumping Crib II.—A final attempt to unwater the combination timber and concrete coffer-dam surrounding Section II of the dock was started on February 6th, 1913. Table 1 gives the depth of unwatering from day to day, as well as the corresponding total rise of crib noted at two points of maximum uplift.

TABLE 1.—Depth of Unwatering of Section II, With Corresponding Total Rise of Crib.

Date, 1913.	Hour.	Depth below.	Rise in Crib, in Feet:	
			At west wall.	At east wall.
Feb. 6		0.00 ft.		
" 11	3.30 P. M.	5.50 "	0.03	0.03
" 12	3.15 P. M.	9.50 "	0.04	0.04
" 13	8.15 A. M.	12.25 "	0.04	0.04
" 13	3.00 P. M.	16.50 "	0.05	0.06
" 14	8.00 A. M.	16.50 "	0.04	0.05
" 14	3.00 P. M.	20.00 "	0.05	0.06
" 15	8.00 A. M.	24.50 "	0.05	0.06
" 15	1.00 P. M.	25.50 "	0.07	0.07
" 16	10.15 A. M.	32.50 "	0.08	0.07
" 16	3.00 P. M.	33.50 "	0.11	0.07
" 17	8.30 A. M.	34.00 "	0.18	0.10
" 17	9.45 A. M.	34.25 "	0.19	0.12
" 17	10.30 A. M.	34.75 "	0.19	0.10
" 17	1.00 P. M	35.50 "	0.25	0.13
" 17	1.45 P. M.	37.05 "		
" 17	2.30 P. M.	36.75 "	0.66	0.15
" 17	2.40 P. M.	36.50 "		0.18
" 17	3.00 P. M.	Coffer-dam failed at about this time.		

The outer row of concrete blocks along each side-wall of the coffer-dam was exposed on February 16th, and the quality of the concrete was found to be excellent. About sixty $2\frac{1}{2}$-in. vertical drain pipes had been placed in the concrete seal during its construction; these drains,

distributed over the area of the coffer-dam, all flowed to some extent when the coffer-dam was pumped out, and a few of them flowed freely; a piece of hose connected with the upper end of one of the free-flowing drains filled with water to within 1 ft. of low-tide level, indicating that the water pressure beneath the concrete seal was nearly equal to sea pressure at corresponding depth, and probably would have been fully equal to sea pressure had the other drainage pipes in the vicinity of the one under test been closed. It will be noted from Table 1 that the rate of uplift of the coffer-dam was not very great until the morning of February 17th, and did not begin to be very alarming until about 1.00 P. M., when it was evident that the work was in grave danger. One of the pumps was stopped at 1.45 P. M. and all pumping was stopped at 2.00 P. M. The coffer-dam collapsed at about 3.00 P. M.; the crib lifted several feet at the time of failure, according to the statements of eye-witnesses.

The portions of the coffer-dam surrounding Sections I and III of the dock failed with the failure of the crib in Section II. Fortunately, no lives were lost, nor was any one injured. Figs. 5 and 6 are photographs showing the work after the collapse. Comparison of soundings taken on the surface of the concrete seal before the collapse with those made 6 weeks thereafter, indicates local vertical displacements varying from a maximum uplift of 9.4 ft. to actual settlements. It has not been practicable to obtain any accurate knowledge regarding the movements in the piling which supported and anchored the seal, but it seemed safe to conclude that it had all been rendered unreliable and therefore practically worthless, and the recently approved plans require its entire renewal.

Pressures.—The weight of excavated material removed from the dock site probably varied from about 2 500 lb. per sq. ft. at the entrance end, where the natural surface was nearly 10 ft. below low-tide level, to about 4 500 lb. per sq. ft. at the inner end of the dock, where the original surface was 14 ft. above tide level. The side slopes of the excavation stood at a very steep angle after the completion of the excavation. These slopes did not seem to be materially disturbed as a result of the coffer-dam, pile-driving, and pumping operations, or by the collapse of the coffer-dam, and there is no evidence of any material subsidence of the side banks. The submerged weight of the concrete in the completed dry dock structure, if considered as uniformly dis-

FIG. 5.—WRECK OF COFFER-DAM AFTER COLLAPSE, PEARL HARBOR DRY DOCK.

FIG. 6.—WRECK OF COFFER-DAM AFTER COLLAPSE, PEARL HARBOR DRY DOCK.

tributed, will be about 2 000 lb. per sq. ft. of area; if the weight of the side-walls is considered separately from the weight of the floor, the unit weight, per square foot of area covered, will be about 3 300 lb. under the side-walls and about 1 400 lb. under the floor. With the dock pumped out and containing a ship of maximum dimensions, the uniformly distributed weight on the underlying soil, in excess of full hydrostatic uplift corresponding to low-tide level, will be about 1 500 lb. per sq. ft.; if the weight of the side-walls is considered separately from that of the floor and ship, the weights imposed under the side-walls and floor will be about 2 300 and 1 100 lb. per sq. ft., respectively.

It is of interest to compare the maximum depths of 37.5 and 37.05 ft. obtained on February 11th, 1912, and February 17th, 1913, in the coffer-dams of Section I and Section II, respectively, with the depth of 38 ft. between extreme high-tide level and the floor of the completed dock. Although the concrete which had been placed in Section I was only 6.52 ft. thick, and in Section II only 6.75 ft. thick, as compared with the thickness of 16 ft. required for the finished dock, and although the side-walls which would tend to anchor the floor and reduce its span had not been started, it will be seen that the partly completed floor in both cases had been called on to resist nearly the maximum uplift which could be brought to bear on the completed structure.

Full hydrostatic uplift on the bottom of the dock at the time of collapse would average about 3 100 lb. per sq. ft. of area. Forces resisting this uplift were developed by the weight of concrete in the seal, the anchorage of the piles, the weight of coffer-dam materials, the weight of water remaining in the coffer-dam, the arch action developed in the concrete seal, and the arch action developed in the layer of soil which was penetrated by the piles; these resisting forces averaged, per square foot, about 1 012 lb. on account of concrete, about 1 080 lb. on account of piles (using 54 lb. per cu. ft. as a conservative estimate for the submerged weight of soil), about 170 lb. on account of coffer-dam, and about 300 lb. on account of water remaining in the coffer-dam; no accurate estimate can be made of the arch resistance in either the concrete or soil; from this analysis it would appear that arch resistance to uplift amounted to about 538 lb. per sq. ft. at the time of collapse.

Although the foregoing analysis based on averages is very interesting, it is extremely doubtful whether the figures given represent the facts due to the following conditions: the dish-shaped form of the concrete seal, which made the unit hydrostatic uplift along the sides much less than under the middle portions; the great differences in the thickness of the concrete blocks which composed the seal and in the length of the anchoring piles; the difference in the depth of water remaining in the coffer-dam at different points, which varied from about nothing at the sides to a depth of several feet over the middle portions; and to the irregular character of the soil underlying the coffer-dam. It is very probable that the collapse of the coffer-dam originated at some particularly weak point in the seal or in the underlying soil, and that local weakness resulted in general failure.

Concrete.

Materials.—The Island of Oahu, unfortunately, does not abound in materials which are best suited for making concrete. Gravel is not obtainable, and there is no silicious sand. A very satisfactory broken stone is made by crushing a hard blue lava rock, but it has been found very difficult to locate a quarry having the necessary capacity and transportation connections, and at the same time being comparatively free from the unsuitable grade of porous rock which is known locally as "puka-puka". There is an abundance of coral and sea-shell beach sand, locally called "Waianae", but its use for concrete is objectionable because of its uncertain chemical stability when incorporated in the completed dock. Screenings from the stone crusher are used to a large extent in lieu of sand. Cement is obtained from California mills.

Sand.—Fig. 7 indicates graphically the character of the natural sands and of the artificial substitute products, obtained either from the stone crusher or by mixing, which were considered as available for use in making concrete. Curve 1 represents a silicious building sand of medium fineness obtained from Puget Sound. Curve 2 represents the screenings obtained through a ¼-in. sieve from the rock crusher which supplies broken stone, and includes crusher dust. Curve 3 represents a combination of the materials used for Curves 1 and 2 mixed in proportion of one part of the former with two parts of the latter. Curve 4 represents crusher screenings through a ¼-in. sieve and is the

same as shown by Curve 2, except that dust passing a 30-mesh sieve is removed. Curve 5 represents coral sand obtained from the vicinity of the work. The mixture represented by Curve 3 was adopted as the best, and was used in making the successful tremie concrete. A combination of equal parts of the materials represented by Curves 4 and 5 shows practically the same characteristics as the mixture represented

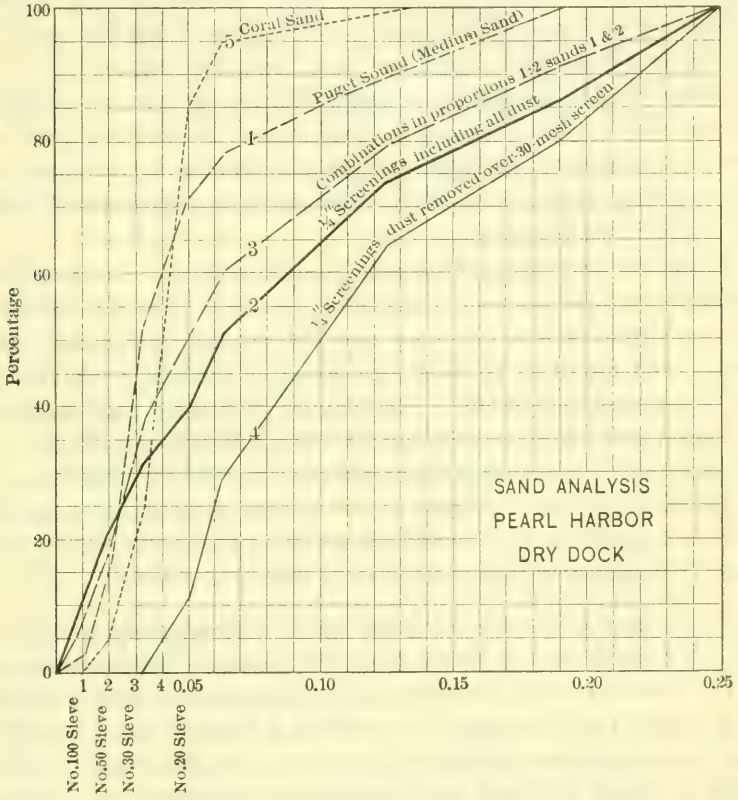


FIG. 7.

by Curve 3 for grains retained on a 20-mesh sieve, but the former contains only 7% of smaller grains which pass this sieve, as compared with 38% of the smaller-sized grains in the latter mixture; the larger percentage of small grains is necessary to obtain concrete of the best quality.

Mixture.—Examination of the concrete which was first placed by the under-water method revealed such serious defects as to render it

practically worthless. There was apparently a lack of bond throughout the mass due to general permeation with sea water, forming a soft, white, chalky substance with the cement which encased the particles of sand and pieces of broken stone in such way as to prevent consolidation. Chemical analysis showed that this chalky substance consisted principally of sulphates. It was evident that salt water would have to be excluded from within the mass of freshly deposited concrete before satisfactory results could be obtained, which required that the concrete mixture would have to be dense and plastic to a high degree.

Briquette tests quickly demonstrated that mortar mixtures containing less than 1 part of cement for each 2 parts of sand could not be used for under-water concrete; briquettes containing a lower percentage of cement swelled perceptibly when placed in either salt or fresh water shortly after mixing.

Numerous experiments were made to determine the mixture which would produce the most dense and plastic concrete. For this purpose, the ingredients in each mix were carefully weighed, then mixed with shovels, and placed in an 8-in. pipe closed at one end, to determine the volume of the mixture; the degree of density was obtained by comparisons of volume with weight. The plasticity was determined by observing the action of the concrete when handled with shovels, and by the settlement into the mass of two wooden rods, having sectional areas of 1 and $2\frac{1}{2}$ sq. in., respectively, when supporting the weight of a man. The results of these tests were in general as follows:

(*a*).—Other conditions being the same, concrete made with 1-in. stone is more dense and plastic than that obtained with larger stone.

(*b*).—Graded stone, containing such percentages of each size that when plotted gave a straight line, produces a concrete which is at the same time most dense and plastic. An excess in the percentage of small pieces of stone increases the plasticity but decreases the density, whereas a deficiency in the small sizes has little effect on the density but reduces the plasticity.

(*c*).—Density and plasticity are a maximum when using Puget Sound sand mixed with screenings which contain crusher dust; elimination of any part of the crusher dust affects the results adversely. One part of Puget Sound sand with from two to three parts of screenings with dust gives the maximum plasticity, but the density is not

affected materially when the proportions are varied between the limits of 1:1 and 1:3.

(*d*).—Mixtures in proportions of 1:2.5:4 are more plastic than those of 1:2:4, but are not as dense; 1:2:4 mixtures are more dense than those of 1:2:3½, but are not as plastic.

Laboratory Tests.—A series of 132 crushing tests were made on 6-in. cubes. Nine sets of twelve blocks were made of concrete mixed in proportions of 1:2:3.5, using for each test uniformly graded stone not larger than 1 in., but with differently mixed or prepared sands for the different sets. The results from these tests are given in Table 2.

TABLE 2.—Results of Crushing Tests.

Class.	Sand.	Age.	Crushing Strength, in Pounds per Square Inch:	
			In salt water.	In fresh water.
I...	Three parts screenings passing ⅛-in. mesh with dust retained on 30-mesh, with 1 part Puget Sound sand.....	3 days	685	620
		28 "	1 435	1 320
		2½ months	1 860	2 285
II....	Screenings passing 3/16-in. mesh with dust retained on 30-mesh.....	3 days	590	470
		28 "	1 280	1 230
		3 months	1 985	1 985
III....	Screenings passing 3/16-in. mesh containing all the dust of fracture.....	3 days	630	565
		28 "	1 505	1 390
		3 months	2 655	2 370
IV....	Sand from Puget Sound.....	3 days	905	845
		28 "	1 745	1 780
		2½ months	2 450	2 360

Table 3 gives the weight per cubic foot of different concretes, as obtained from laboratory blocks. For brevity, cr. will be used to designate crusher screenings, P. S. for sand from Puget Sound, and W. for Waianae or local coral sand.

Field Tests.—Fifteen large-sized test blocks were made under conditions which resembled those actually existing when placing underwater concrete for the bottom of the dock. The concrete for these tests was deposited with tremies in water about 52 ft. deep. The blocks were about 4.5 ft. wide and 6.5 ft. long. The end of the tremie was held near one corner of the form so that concrete had to flow to an extreme distance of about 6 ft. after leaving the tremie. A 2-cu. yd. Ransome mixer was used for the concrete, which was made very wet, but with water limited so that it would not rise to the surface during transportation from mixer to tremie. The results from

TABLE 3.—Weight of Concrete per Cubic Foot.

Block No.	Mix.	Sand.	Stone.	Condition.	Weight.
501.....	1 : 3.6 : 6.5	2 cr ½ in. to dust 1 P. S.	2½ in. to ¼ in., graded..	24 hours moist air.....	148.75
501.....	"	"	"	16½ hours, water.....	151.25
501.....	"	"	"	2 days, air.....	148.125
501.....	"	"	"	3 " "	147.5
502.....	"	"	"	24 hours, moist air.....	151.25
502.....	"	"	"	16½ hours, water.....	151.25
502.....	"	"	"	2 days, air.....	152.5
502.....	"	"	"	3 " "	152.5
9-D..	"	cr. ½ in. to dust.....	2½ in. to ¼ in., ungraded	Wet.....	137.5
10-D..	"	3 cr. ½ in. to dust 1 W..	"	"	146.5
11-D..	"	2 cr. ½ in. to dust 1 W..	"	"	150.0
12-D..	"	3 cr. ½ in. to dust 2 W..	"	"	147.6
21-D..	"	cr. ¼ in. to dust.....	"	"	147.7
22-D..	"	3 cr. ¼ in. to dust 1 W..	"	"	149.7
23-D..	"	2 cr. ¼ in. to dust 1 W..	"	"	150.5
24-D..	"	3 cr. ¼ in. to dust 2 W..	"	"	150.0
33-D..	"	cr. 3/16 in. to dust.....	"	"	157.0
34-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	158.2
35-D..	"	2 cr. 3/16 in. to dust 1 W..	"	"	157.0
36-D..	"	3 cr. ¼ in. to dust 2 W...	"	"	149.8
36-1 D.	"	3 cr. 3/16 in. to dust 2 W..	"	"	153.0
42-D..	"	3 cr. ½ in. to dust 1 W..	2½ in. to ¼ in., graded..	"	155.0
45-D..	"	cr. ½ in. to dust.....	"	"	153.0
46-D..	"	3 cr. ½ in. to dust 1 W..	"	"	160.0
47-D..	"	2 cr. ½ in. to dust 1 W..	"	"	160.5
48-D..	"	3 cr. ½ in. to dust 2 W..	"	"	161.0
56-D..	"	3 cr. to dust 2 W.....	"	"	150.3
57-D..	"	cr. ¼ in. to dust.....	"	"	151.0
58-D..	"	3 cr. ¼ in. to dust 1 W..	"	"	154.0
59-D..	"	2 cr. ¼ in. to dust 1 W..	"	"	155.0
67-D..	"	2 cr. 3/16 in. to dust 1 W.	"	"	154.8
69-D..	"	cr. 3/16 in. to dust.....	"	"	148.0
70-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	154.0
72-D..	"	3 cr. 3/16 in. to dust 2 W..	"	"	150.3
79-D..	"	cr. ½ in. to dust.....	"	"	145.2
80-D..	"	cr. ¼ in. to dust.....	"	"	148.7
81-D..	"	cr. 3/16 in. to dust.....	"	"	153.0
503.....	1 : 3 : 6.5	2 cr. ¼ in. to dust 1 P. S.	"	24 hours, moist air.....	153.125
503.....	"	"	"	16½ hours in water.....	153.75
503.....	"	"	"	2 days in air...	150.62
503.....	"	"	"	3 days in air...	153.75
504.....	"	3 cr. ¼ in. to dust 2 P. S.	"	24 hours, moist air.....	152.5
504.....	"	"	"	16½ hours, water.....	151.25
504.....	"	"	"	2 days in air...	151.87
504.....	"	"	"	3 days in air...	151.87
1-D..	"	cr. ½ in. to dust.....	2½ in. to ¼ in. ungraded	Wet.....	136.1
3-D..	"	2 cr. ½ in. to dust 1 W..	"	"	146.3
4-D..	"	3 cr. ½ in. to dust.....	"	"	145.3
13-D..	"	cr. ¼ in. to dust.....	"	"	140.0
14-D..	"	3 cr. ¼ in. to dust 1 W...	"	"	149.0
15-D..	"	2 cr. ¼ in. to dust 1 W...	"	"	151.5
25-D..	"	cr. 3/16 in. to dust.....	"	"	149.5
26-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	154.0
27-D..	"	2 cr. 3/16 in. to dust 1 W..	"	"	153.1
32-D..	"	3 cr. 3/16 in. to dust.....	"	"	155.0
37-D..	"	cr. ½ in. to dust.....	2½ in. to ¼ in. graded...	"	152.0
38-D..	"	3 cr. ½ in. to dust.....	"	"	154.0
39-D..	"	2 cr. ½ in. to dust 1 W..	"	"	154.5
40-D..	"	3 cr. ½ in. to dust 2 W..	"	"	154.0
49-D..	"	cr. ¼ in. to dust.....	"	"	143.0
50-D..	"	3 cr. ¼ in. to dust 1 W..	"	"	150.0

TABLE 3.—(*Continued.*)

Block No.	Mix.	Sand.	Stone.	Condition.	Weight.
51-D..	1 : 3 : 6.5	2 cr. ¼ in. to dust 1 W...	2½ in. to ¼ in. graded...	Wet.....	146.5
61-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	142.7
62-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	149.5
63-D..	"	2 cr. 3/16 in. to dust 1 W..	"	"	149.0
64-D..	"	3 cr. 3/16 in. to dust 2 W..	"	"	148.0
73-D..	"	cr. ½ in. to dust.....	"	"	140.0
74-D..	"	cr. ¼ in. to dust.....	"	"	149.0
75-D..	"	cr. 3/16 in. to dust.....	"	"	151.0
547.....	1 : 2 : 3.5	2 cr. ¼ in. to dust 1 P. S.	.....	"	151.25
547.....	"	"	.....	24 hours in air.	150.625
547.....	"	"	.....	16½ hours, water.....	152.5
547.....	"	"	.....	2 days in air...	152.5
547.....	"	"	.....	3 days in air...	152.5
505-D..	"	"	.....	24 hours moist air.....	153.7
505-D..	"	"	.....	16½ hours, water.....	152.5
505-D..	"	"	.....	2 days in air...	151.87
505-D..	"	"	.....	3 days in air...	153.75
506-D..	"	"	.....	Wet.....	151.0
507-D..	"	1 3/16 washed, cr. 1 P. S...	.....	"	155.0
508-D..	"	2 pt. fine, cr. 1 P. S.....	.....	"	157.0
509-D..	"	2 cr. 3/16 in. to dust 1 W..	.....	"	149.3
510-D..	"	cr. 3/16 in. to dust 1 P. S..	.....	"	153.2
511-D..	"	P. S.....	.....	"	151.5
512-D..	"	cr. 3/16 in. to dust.....	.....	"	148.0

eight of these large test blocks, and also from small cubes made from the same mixes which were seasoned in the laboratory, are given in Table 4.

The concrete in Block No. 2 was fairly smooth-flowing and plastic. The surface of the block when raised for examination was rough, but hard and compact.

The concrete in Block No. 9 was free-flowing, sticky, and generally excellent in appearance. When the block was removed from the water it was found that it had perfectly filled the form, and that the grain of the wood in the form was distinctly reproduced in the sides of the block; the top of the block was flat and nearly level. In short, this specimen was the equal of high-grade concrete placed under favorable conditions on shore. The concrete in this block weighed 151.2 lb. per cu. ft.

The concrete in Block No. 12 did not show any serious separation of materials, but it was not of uniform texture, and had not hardened satisfactorily after an immersion of 24 days. It weighed 147 lb. per cu. ft.

TABLE 4.—Results of Tests of Large Test Blocks.

No.	Diameter of tremie, in inches.	Mix.	Sand.	Stone.	Age, in days.	Crushing Strength, in Pounds per Square Inch.	
						Tremie block.	Laboratory block.
2	15	1 : 2 : 3½	3/16-in. screenings retaining dust	1-in. well graded	30 36 90	860	1 355 2 030
9	15	1 : 2 : 3½	2 parts ¼-in. screenings retaining dust, 1 part Puget Sound	1-in. well graded	30 90 120	2 075 ..... 2 113	1 960 2 655 .....
12	15	1 : 2 : 3½	1 part 3/16-in. screenings and dust passing 30-mesh removed. 1 part Puget Sound	1-in. well graded	24	1 205	None
10	15	1 : 2 : 3½	1 part 3/16-in. screenings containing dust. 1 part Puget Sound	1-in. well graded	105	2 485	None
11	15	1 : 2 : 3½	Puget Sound	1-in. well graded	16 30 75 28	1 325 1 670 2 350	1 405 1 700 2 410 1 040
3	18	1 : 2 : 3½	2 parts ¼-in. screenings containing dust. 1 part coral.	1-in. well graded	38 68 90	1 010 2 030	.... 1 570
8	18	1 : 3 : 5	2 parts ¼-in. screenings containing dust. 1 part coral.	1½-in. not graded	....	No	tests
13	15	1 : 2 : 4	Puget Sound	1½-in. not graded	....	No	tests

The concrete in Block No. 10 was very free-flowing and plastic, and presented a good appearance when raised for examination.

The concrete in Block No. 11 very much resembled that in Block No. 10, except that it was a little less plastic. It weighed 149.4 lb. per cu. ft.

The concrete in Block No. 3 was free-flowing and plastic, but it was found that there was considerable washed stone in the bottom of the form, which had accumulated during the early stage of the pouring, and that there were soft pockets in the block where the coral sand had segregated excessively.

The concrete in Block No. 8 was ashy in appearance, and was not plastic. The block was very inferior, and the ingredients were badly washed and separated.

The concrete in Block No. 13 was mixed moist, but not wet, with the result that the tremie choked repeatedly; the concrete did not flow smoothly to fill the form, and sea water entered, not only into the deposited mass, but also into the tremie; the result was failure, as far as obtaining a satisfactory concrete was concerned.

The mixture and method of placing the tremie concrete used in the coffer-dam surrounding Section II were the same as used for Block No. 9.

Tremies.—Approximately, 14 840 cu. yd. of concrete were deposited through tremies in water varying in depth from about 54 to 44 ft. A great deal of interesting experience in the use of tremies has been incidentally acquired, due to the fact that the product, which at first was practically worthless, was gradually improved until finally it attained a high degree of excellence. This improvement followed careful experiments with tremies of various diameters, manipulated in various ways, and handling concrete mixed in many different proportions with variously graded and selected materials. Tremies 12, 15, and 18 in. in diameter were used in these experiments, and the results seem to justify the following general conclusions:

(*a*).—The results from a tremie depend to a great extent on the proportions, character of materials, and plasticity of the concrete which is being used.

(*b*).—The excessive frictional resistance to the movement of concrete in a 12-in. pipe causes frequent clogging in the pipe and greatly reduces the pressure at the exit. Under these conditions, it becomes impracticable to hold the end of the tremie embedded in the deposited concrete, thereby bringing all newly placed concrete into direct contact with sea water, with consequent injury to the entire product.

(*c*).—The frictional resistance to the movement of concrete in an 18-in. pipe is not sufficient to prevent the occasional loss of a charge in the tremie, thereby interrupting the filling of a form, with the added uncertainty as to the quality of the product. There is no difficulty in maintaining the end of this tremie embedded in concrete, and readily filling the form without any disturbance of the mass which would admit sea water.

(*d*).—The frictional resistance in a 15-in. tremie is apparently just about right to obtain the proper discharge pressure necessary for efficiently regulating the flow of concrete by raising and lowering the tremie with the end maintained within the deposited mass. No difficulty is experienced, when using this tremie, due to short interruptions in the supply of concrete from the mixer. Concrete discharged from this

tremie flows freely to distant parts of the form, without causing disturbance in the mass which would admit sea water, and without any tendency for sea water to rise in the tremie.

(*e*).—A tremie 15 in. in diameter is best suited for the work, and was adopted and actually used for placing the greater part of the tremie concrete.

The 15-in. tremie consisted of a riveted steel pipe, with a funnel-shaped steel hopper, about 5 ft. in diameter, bolted to the upper end. The tremie hopper was suspended from a timber storage hopper having a capacity of about 2 cu. yd., and fitted with a 12-in. valve in the bottom for controlling the flow of concrete into the tremie. Concrete was mixed in a 2-cu. yd. mixer at the rate of from 40 to 50 batches per hour, each batch producing about 1.85 cu. yd. Concrete from the mixer was discharged into 2-cu. yd. bottom-dump tubs, which were transported on flat cars hauled by cable to a derrick, thence into the tremie storage hopper. When beginning operations, the tremie was set with its lower end resting on the bottom of the form and then raised from 2 to 3 in. A bag loosely filled with straw was then placed in the tremie hopper to serve as a piston which descended under the burden of concrete to force the sea water out of the tremie. It was customary to assemble about 6 cu. yd. of concrete at the derrick before starting to fill the tremie, as it was found that this quantity when rapidly poured was sufficient to charge the tremie fully and embed its end to a depth of from 1.5 to 3.0 ft., and effectually seal the tremie from the entrance of sea water.

Special effort was made, after the tremie was first properly charged, to supply the concrete required to fill the form quickly and regularly; by carefully manipulating the depth of the tremie to correspond with its supply of concrete, it was found that the surface of the concrete in the form could be maintained in an almost flat and level condition, with uniform displacement throughout the form, and with the exposure throughout the process of only one unvarying surface to the injurious effect of contact with sea water. The forms for tremie concrete were each about 12 ft. square, and were of rough 1-in. boards and 2 by 4-in. studding; after framing the pieces for one side of a form on the bank, it was sunk in a vertical position to its proper place; the sides of the form were then closely joined together and braced by a diver, who closed any openings in the bottom with concrete in bags and also made

sure that the form was properly cleared and ready for concrete just before pouring was begun. Approximately 40 cu. yd. of concrete were required to fill one form.

General Tests and Investigations.

Borings.—Numerous borings have been made at the dock site to determine sub-surface conditions. These consist of a series of 38 borings of the wash type which were made in 1908, before work on the dock was started, and a series of 18 of the core type made in 1913.

The preliminary borings were on the axial line of the dock, and on two parallel lines about 65 to 75 ft. on each side of the axis. The borings were at intervals of about 100 ft. on these lines, and were arranged so that there was one on each side line opposite one on the center line, to facilitate the making of cross-sections. The original ground level over the dock site varied between an elevation of 14 ft. above mean low-tide level to 8 ft. below. The preliminary borings were carried to about the same average depth of 75 ft. below tide level, or about 23 ft. below the bottom of the dock. The borings were made by driving a casing pipe into the ground, loosening the penetrated material by blasting and churning when necessary, and then removing the contents of the casing with a water jet from a small pipe within. The only samples of the underlying soil which could be obtained by this method were the deposits found in the bottom of a bucket in which the overflow from the casing was collected; due to the disintegration of the material and the removal of the lighter particles which this process involved, there was necessarily a considerable difference between the samples and the original formations which they represented.

There were several special reasons for undertaking the second series of borings made in 1913:

(*a*) To check the results found by the 1908 series.

(*b*) To find, if possible, an explanation for the peculiar movements of the coffer-dam on the three occasions when efforts were made to unwater the excavation.

(*c*) To determine more definitely the suitability of the site for the construction of a graving dock.

(*d*) To determine the weight of the materials in the various strata penetrated.

The new borings, therefore, were located near the points which had been selected for the preliminary borings, were carried to considerably greater depths than those reached by the earlier series, and were made by methods which permitted of obtaining much more accurate information regarding the materials penetrated.

The first fourteen borings of the 1913 series were made by using a 6-in. casing pipe in combination with the churning and jet devices which comprise the usual wash-boring outfit; a heavy driving rod fitted at the lower end with a short removable piece of 3-in. pipe or core barrel was utilized for obtaining samples. With this equipment it was found possible to drive the core barrel to undisturbed depths below the bottom of the casing, and obtain cores as frequently as desired, provided the material was of a nature or consistency which would remain in the core barrel while being raised. The core barrel was provided with holes above the highest point reached by the cores to prevent the forming of internal water pressure by an entering core which would tend to disintegrate or modify the character of the core when it was withdrawn. These fourteen borings ranged in depth from 84 to 192 ft., with an average of 144 ft. below low-tide level. It was found in practice that considerable force was frequently required to remove cores from the core barrel, and that probably the physical character, and especially the density, of the material was thereby affected.

The last four borings of the 1913 series were made with a rotary Calyx drilling outfit having dimensions for obtaining 2-in. cores. Cores obtained with this apparatus were not compressed or modified while being secured, and more truly represented the original formations than those obtained with the driving rod. These four borings reached depths varying from 94 to 107 ft., with an average depth of 99 ft., below low-tide level.

While certain borings were in progress, and when the top of the casing tube was only a short distance above sea level, it was noted that there was an overflow of fresh water from the casing; by increasing the elevation of the top of the casing it was found that the height to which the fresh water would rise could be computed by multiplying the depth of the water-bearing stratum by the ratio between the weights of salt and fresh water; in other words, the phenomenon was

not of Artesian origin, but indicated the presence of ground-water below an impervious stratum.

With due allowance for the very irregular character of the subsoil, the results from the 1913 series of borings were in close accord with those found from the preliminary borings in 1908. The mud stratum beneath the dock is apparently impervious and unbroken in its continuity, but varies in thickness from about 4 to 40 ft. The borings gave no evidence of unusual compressibility or of fluidity in any of the underlying formations.

The character of the mud stratum has been of vital importance throughout this work. The original design for the dock and the method selected by the contractor for its construction were undoubtedly based on the belief that the stratum was continuous, solid, impervious, and comparatively non-compressible. The stratum apparently has all these qualities in its original form, but it is probable that the conditions of solidity and imperviousness were impaired by building operations. In the writer's opinion, all the trouble which has been experienced since the work was started may be attributed to insufficient solidity of this stratum due to thin portions and to the breaking of the stratum by piling and by upheavals after removing the top cover. Figs. 8 and 9 are photographs of cores obtained from borings through the mud stratum; the plasticity of the material is indicated by the bands on the sample shown in Fig. 8, which formed when the core was driven out of the core barrel; Fig. 9 illustrates the tendency of the mud to check and crack when it dries. Samples of the mud when freshly obtained are plastic like putty or clay, but when immersed in water quickly and completely disintegrate like so much corn-meal. A sample placed on the tongue does not give any impression of grit. It is interesting to note that it is this material which forms the soil in which sugar cane thrives so wonderfully.

Bearing Tests.—Two tests were made to determine the bearing value of the soil in the bottom of the dry dock excavation. These tests were conducted before any piling had been driven for the support of the dock and after the excavation had been practically completed to the required depth at the points selected for the tests.

The first test, made in April, 1910, covered an area of 4 sq. ft. Under a load of 1.5 tons per sq. ft., there was a settlement of about 3 in. in 12 hours; the greater part of this settlement occurred immedi-

ately after the application of the load, as might have been expected with a load applied to soft ground. The load was then gradually increased, during a period of 25 min., until it amounted to 3 tons per sq. ft., under which the total settlement reached 5.25 in. after 29 hours, and was slowly increasing.

The second test was conducted in May, 1910, and was applied to a strongly trussed tripod having legs 10 ft. apart, each resting on a shoe having an area of 4 sq. ft. After 42 hours under a load of 1 ton per sq. ft., the settlements under each shoe were 2, $3\frac{3}{4}$, and $2\frac{1}{4}$ in., respectively. The load was gradually increased to 4 tons per sq. ft., which increased the settlements to $4\frac{1}{2}$, $9\frac{7}{8}$, and 5 in., respectively. After the load of 4 tons per sq. ft. had been supported for 24 hours, the settlements had increased to $4\frac{7}{8}$, $11\frac{1}{8}$, and $5\frac{3}{8}$ in.; after a further period of 17 hours under this load, the settlements had increased only $\frac{1}{100}$, $\frac{1}{100}$, and $\frac{1}{50}$ in., which are too small to be measured accurately with ordinary field instruments and for all practical purposes the tripod could be considered as having reached a condition of rest. The possible unequal distribution of the load to the three legs probably accounts in part for their unequal settlement, but the real cause of the inequality is undoubtedly due to a difference in the bearing value of the soil at the three points.

Pile Tests.—During May, 1911, five test piles were driven in the bottom of the coffer-dam surrounding Section I, and fifteen piles at various points in the bottom of the excavation outside of the coffer-dam. The penetration of the piles varied from 18.2 to 48.9 ft. After flooding the coffer-dam it was found that one of the test piles inside the coffer-dam had settled $\frac{3}{8}$ in., two had settled $\frac{1}{8}$ in., and one had remained without change; the record of the fifth pile has been lost. One of the piles inside the coffer-dam, with a penetration of 27.2 ft., was gradually loaded during a period of 3 hours with a load of 16 tons; the pile settled a total of $1\frac{5}{8}$ in. while being loaded, and, after carrying the load for 44 min., had gone down an additional $\frac{1}{8}$ in.; no more settlement had occurred 70 min. later.

Four tests were made during May, 1911, to determine the anchorage value or resistance to pulling of a single pile. The test piles had been driven in the bottom of the dock excavation, and were separated by considerable distances. It was found to be impracticable to apply

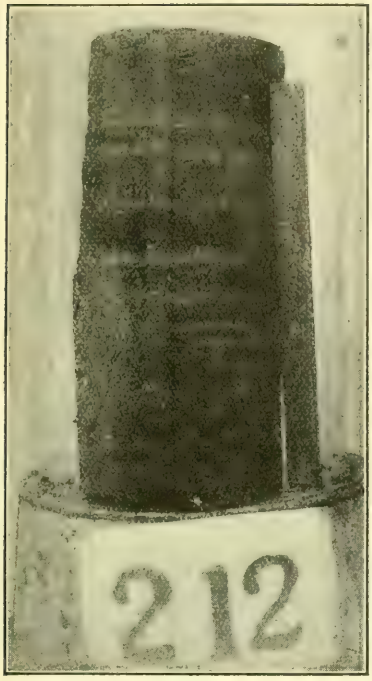


FIG. 8.—MUD CORE, SHOWING BANDS ON SURFACE WHEN DRIVEN OUT OF CORE BARREL.

FIG. 9.—MUD CORE, SHOWING CRACKS AND CHANGES IN APPEARANCE AS RESULT OF DRYING.

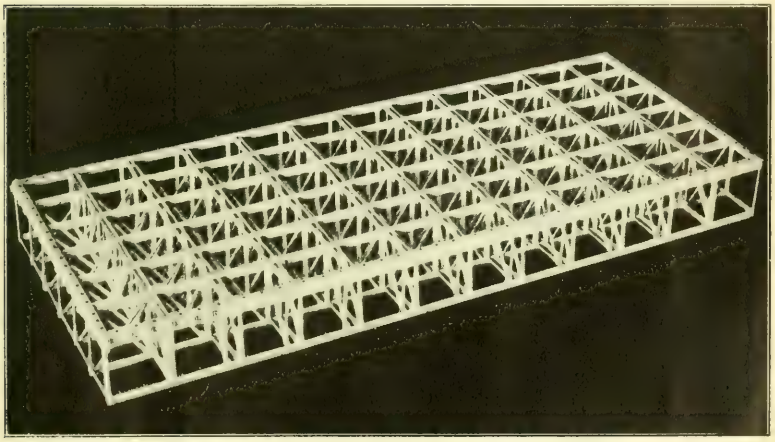


FIG. 10.—STEEL FRAME TO BE EMBEDDED IN BASE OF DOCK SECTION.

loads sufficient to draw the piles; the results of the tests are given in Table 5.

TABLE 5.—Results of Tests in Pulling Piles.

Pile No.	Penetration.	Pull.		Rise, in feet.	Permanent set.
		Tons.	Time, in hours.		
7	19.3 ft.	56	18	0.002	....
9	32.2 "	40	18	0.017	0.00
14	31.4 "	40	18	0.002	0.00
19	36.9 "	40	18	0.003	0.00

It is very probable that the "rise" of the piles under load was apparent rather than real, in view of the small movements noted and the complete recovery after removing the load, and were due either to observation errors or a slight straightening of the piles while under pull.

A test was made during February, 1913, to determine the pull which would be required to draw a pile from a block of concrete in which the head of the pile had been embedded. The pile was 45 ft. long, 12 in. in diameter at the butt, and 7.5 in. in diameter at the point. The pile was suspended in water with the butt pointing downward into a form resting on the bottom of the dock; concrete mixed in the proportions of 1:2:3.5 was poured into the form through a tremie until the head of the pile had been embedded to a depth of 5 ft. A direct and slowly increased pulling load was applied to the pile after the concrete had set for 31 days; when the pull amounted to 29.2 tons, the upper end of the pile broke off without any movement of the pile head having been noted; after getting a new hold on the pile the load was again gradually applied until it amounted to 47.0 tons, but without budging the pile head in the concrete.

During May and June, 1913, a loading test was made on a group of piles which had been driven in the bottom of the dock excavation. It had been planned to carry a grillage platform on four rows of five piles and apply the load on an upper deck above sea level, which was to be supported on the grillage by a braced trestle. Due to irregularities in spacing the piles in a depth of about 52 ft. of water, it was found possible to bring only fifteen of the piles to bear under the

grillage. The loading was begun on May 20th, and the load was gradually increased until May 31st, when it amounted to 400 tons; the load was allowed to remain until June 10th, and was then gradually removed during the next 4 days. The settlement slowly increased while supporting the fixed load, and amounted to 5.0, 1.1, 2.45, and 2.25 in., respectively, at the four corners of the platform at the time the unloading began. After the removal of the load there was a rise at the four corners of 3.25, 0.35, 1.25, and 1 in., respectively. After removing the trestle and grillage, it was found that the timber was badly crushed at various places, and there was evidence of deflection in the uprights; these conditions combined would probably account for a considerable part of the recorded settlements. This test applied an average load of about 26.7 tons to each of the fifteen piles, but, due to the crushing of the timber, there is much uncertainty as to the actual distribution of the load and the settlement of the piles under load. Observations on unloaded piles in the vicinity of the grillage showed settlements running up to 0.4 in., as the load was applied to the platform, but the latter was permanent after the removal of the load; these settlements would indicate a compression of the ground below the points of loaded piling.

During August and September, 1913, a second loading test was made on a group of twenty piles, in much the same manner as that followed during the preceding test, except that levels were taken on splines attached directly to the piles, and precautions were taken to eliminate errors which would result from the crushing or yielding of the temporary framework. Table 6 shows the results from this test.

These results compare very closely with those obtained from the preceding test.

The specification for pile-driving provided for the use of a water jet, if directed by the officer in charge. In view of the impossibility of obtaining pile penetrations as great as were anticipated and as were desired for the adopted constructive methods, a special test was conducted during April, 1913, to determine the local practicability and value of the jet process. The test pile was 76 ft. long, 15 in. in diameter at the butt, and 9.5 in. in diameter at the point, and was driven approximately in the center of the dock excavation, in water 52 ft. deep. After 80 min. of jetting through a 2½-in. pipe, with a

TABLE 6.—Loading Tests of Piles.

Date, 1913.	Load period, in days.	Average load per pile, in tons.	Total settlement, in inches. Average of 20 piles.
August 29th	0	2	0.00
September 3d	5	8	0.42
" 4th	6	9	0.48
" 5th	7	11	0.55
" 6th	8	15	0.84
" 7th	9	15	0.97
" 8th	10	15	1.03
" 9th	11	17	1.20
" 10th	12	19	1.09
" 11th	13	20	1.16
" 12th	14	20	1.22
" 13th	15	20	1.25
" 14th	16	..	....
" 15th	17	22	1.25
" 16th	18	24.3	1.28
" 17th	19	24.3	1.61
" 18th	20	24.3	1.53
" 19th	21	24.3	1.59
" 20th	22	24.3	1.64
" 21st	23	24.3	1.61
" 22d	24	24.3	1.69
" 23d	25	24.3	1.75
" 24th	26	24.3	1.77

$\frac{3}{8}$-in. nozzle, under a pressure varying from 150 to 350 lb. per sq. in., the pile had sunk 2.1 ft. The pile was then struck 35 blows with a 5-ton. 42-in. stroke, steam hammer, which added 4.3 ft. to the penetration. With difficulty the pile was then drawn and the jet was inserted in the hole and churned for 45 min., with water pressure at 350 lb., with the result that the hole was filled to a depth of 4.4 ft. with loosened coral rock which the jet could not remove. The jet was then taken out and the pile re-inserted and alternately driven, pulled, and re-driven, until it reached a penetration of 12 ft. The pile was then again pulled and inserted with the jet pipe, with the nozzle at the point of the pile; a jet was applied at 350 lb. pressure, and the pile came to rest at a penetration of 5.5 ft.; after continuing the jet for 15 min., with the weight of the hammer resting on the pile, the penetration had increased $\frac{1}{2}$ ft. An effort was then made to drive the pile while the jet was in operation, with the nozzle held at the pile point, but after eight blows the nozzle became clogged and the jet was removed. The alternate driving and pulling of the pile was then resumed, with gains of about 12 in. in penetration for each twelve blows, until a total penetration of 14 ft. had been obtained, when further operations were suspended because of the great difficulties attending the pulling process.

Caisson, Pumps, and Capstans.

Only brief mention will be made of the caisson, pumping equipment, and electric capstans which, although necessary accessories, do not form part of the graving dock structure proper and are furnished under separate contracts.

The caisson is of the floating boat type, and is equipped with electrically operated valves, pumps, and capstans. Electricity for the operation of the caisson will be obtained through portable connections with the station service. The caisson, built and delivered by the Union Iron Works, of San Francisco, has cost $133 923.93, and, properly modified to suit different tidal conditions, is almost identical in design with the caissons provided for Dock No. 4 at New York and for Dock No. 2 at Puget Sound.

The pumping equipment comprises four main discharge pumps and two drainage pumps. All pumps are of the vertical-shaft, volute type, each operated by a direct-connected, 3-phase, 2 200-volt, induction motor. The main pumps operate at 219 to 222 rev. per min., have a 54-in. suction, 48-in. discharge, and deliver approximately 8 000 cu. ft. per min. against 32 ft. head; 10 600 cu. ft. at 20 ft. head; 11 600 cu. ft. at 10 ft. head; and 11 800 cu. ft at zero head; the approximate input into the motor corresponding with this service is 700 h.p. at 32 ft. head; 586 h.p. at 20 ft. head; 475 h.p. at 10 ft. head; and 360 h.p. at zero head; considering that a motor and pump, together with its suction and discharge friction, combine to form a unit, the efficiency of a unit based on this performance is approximately 71.4% at 32 ft. head, 70.2% at 20 ft. head, and 47.4% at 10 ft. head. The drainage pumps operate at 585 rev. per min., having 20-in. suction and discharge, and, when operating against a head varying from 37.5 to 42.5 ft., deliver approximately from 675 to 560 cu. ft. per min. with an input varying from 74.7 to 71.7 h.p., or with a combined efficiency varying from 66.3 to 64.4 per cent. The pumps are below the floor of the dock thereby eliminating all priming troubles. The motors are on a floor above tide level. The weight of the rotor, shaft, and impeller, of the main pumps, is carried by a collar bearing, just below the motor. The journals, bearings, and steady rests required for the pumping equipment are lubricated by a continuous system, through which oil is circulated under a pressure of about 180

lb. per sq. in. The pumping equipment for the Pearl Harbor dock is practically identical with that which has been installed in connection with Dry Dock No. 4 at New York and Dry Dock No. 2 at Puget Sound, and the foregoing operating data are based on results actually obtained during the operation of those docks.

The dock is to be equipped with nine capstans, one at the head of the dock and four along each side. These capstans will be used for hauling a vessel into the dock and for locating and centering the vessel correctly with reference to the keel blocks which have been prepared for its support. Each capstan is operated by a 50-h.p., 220-volt, induction motor, and the apparatus is designed to give a rope pull of 35 000 lb. at the rate of 12 ft. per min., or of 14 000 lb. at the rate of 30 ft. per min. These capstans are identical with those at Dry Dock No. 4 at the Navy Yard, New York.

Suggested Plans.

The long series of troubles and difficulties which had marked the progress of field work, and which culminated in the collapse of the coffer-dam on February 17th, 1913, naturally led to serious thought being given to other methods of construction and changes in plans which would tend to avoid building hazards and insure successful results.

Cox Plan.—L. M. Cox, M. Am. Soc. C. E., Civil Engineer, U. S. N., suggested two different schemes for the work. The first was presented in 1911, and contemplated the adoption of a sectional method of construction in which the units would consist of boats or structures which would be partly constructed on shore and then floated to position and sunk in a manner very similar to that independently suggested at later dates by Mr. Harris and Mr. Noble.

Mr. Cox's second scheme, submitted during the early part of 1914, urged the practicability of constructing the dock by the general methods and according to the general plans which were being followed at the time of the collapse of the coffer-dam by utilizing a ballast tank to assist in resisting hydrostatic uplift on the bottom of the unwatered coffer-dam. The dimensions of the proposed tank were to be as great as practicable, within the limits of the clear space between the side-walls of the completed dock and the end bulkheads of the

coffer-dam. The tank was to have a substantial interior framework, designed to enable it to carry its full capacity of sea water when resting on a grillage support on the floor of the coffer-dam and at the same time to serve as part of the coffer-dam bracing. The primary function of the tank was to provide a ready means of applying a temporary ballast or loading to hold down the tremie-placed concrete bottom of the unwatered coffer-dam during the period required to complete the concrete work in the dock floor and side-walls. Work under the tank would be conducted in alleyways under the structure; these alleyways would first be arranged through the supporting grillage in which concrete would be placed to the required height and on which the tank would in turn be supported while the grillage was removed and concrete was placed in the vacated spaces. After the completion of the concrete work within any coffer-dam, the latter would be flooded and the tank emptied and floated through a gateway in the coffer-dam bulkhead and placed in readiness for use in the next adjacent coffer-dam.

Gayler Plan.—Recommendations for changes were submitted immediately after the collapse by E. R. Gayler, M. Am. Soc. C. E., Civil Engineer, U. S. N. This plan proposed to construct the dock of large concrete blocks joined together with tremie concrete and with steel reinforcement. Each block would weigh about 150 tons, and would be made on the surface of the ground in the vicinity of the dock. The plan had its inception in the fact that a floating crane having a capacity for handling loads of 150 gross tons had been provided as part of the station's equipment, and could be made serviceable for placing the blocks. Any upheaval of the dock during construction or after completion was to be avoided by increasing the thickness of the floor, thus not only adding to the strength of the floor, but incidentally making the weight of the dock greater than any possible hydrostatic uplift. All foundation piling would be carefully cut off, and the bottom would be prepared to the correct form before beginning to place the blocks. Spaces about 2 ft. wide would be left between adjacent blocks, except where longitudinal reinforcing trusses were to be placed, where the spaces would be about 6 ft. wide; these spaces would be filled later with tremie-placed concrete. Anchorages for transverse reinforcement would be provided in specially prepared

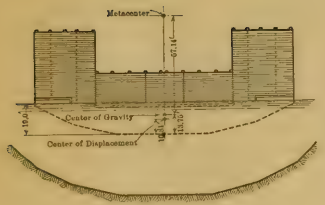


FIG. 1
CAISSON AFLOAT READY TO SINK

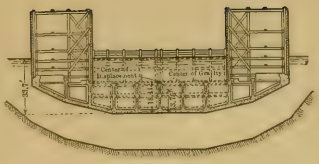


FIG. 2
CENTRAL PORTION FILLED WITH WATER TO SEA LEVEL

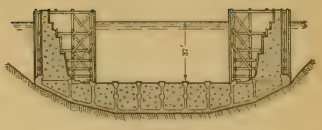


FIG. 3
SIDE-WALL COMPLETED

NOBLE DESIGN "TYPE A"

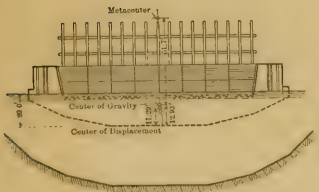


FIG. 1
CAISSON FLOATED, DRAFT 20 FT.
Fill 4½ ft. of water in 28 center pockets before floating, to reduce bending

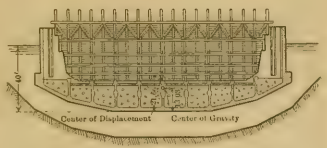


FIG. 2
DRAFT 40 FT.
Build up side-wall pockets to next step, place 6½ ft. of concrete in pockets under walls and 5 ft. additional concrete in pockets of the two middle rows

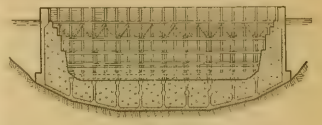


FIG. 3
ALL CONCRETE IN PLACE

NOBLE DESIGN "TYPE B"

GAYLER BLOCK TYPE

SKETCHES ILLUSTRATING METHODS OF CONSTRUCTION PROPOSED BY CIVIL ENGINEER GAYLER, U.S.N. AND MR. ALFRED NOBLE

anchor-blocks; after placing the anchor-blocks the connecting steel would be laid in the open spaces between the blocks and in grooves formed in the intermediate blocks; each transverse chain or tie would include a turnbuckle which would be tightened by a diver before embedding it in the concrete. Unequal settlements and cracks after the completion of the dock would be prevented by first allowing each block to settle independently to a position of rest before bonding it with its adjacent blocks, then combining the blocks in the side-walls, to permit the independent adjustment of the walls as a whole to positions of rest, then bonding the floor blocks with each other and with the side-walls, with the adjustment of the transverse tension bars. Plate XXII shows the general dimensions and arrangements proposed by this plan.

Harris Plan.—F. R. Harris, M. Am. Soc. C. E., Civil Engineer, U. S. N., submitted two plans in a letter dated September 26th, 1913. One plan was based on continuing the work in accordance with the plans and method which were being followed at the time of the collapse of the coffer-dam, but proposed to obtain stability while the coffer-dam was unwatered by increasing the quantity and distribution of the tremie-placed concrete in the seal of the coffer-dam.

Mr. Harris' second plan proposed the adoption of a sectional-boat method, like that suggested by Mr. Cox in 1911. Mr. Harris recommended that the length of the dock should be separated into ten portions, approximately 99 ft. long, measured longitudinally with the dock. The base or hull for each unit was to be made of 1:2:4 reinforced concrete, and was to be constructed on launching ways. The bottom of the base was to be flat, thus differing from the dish-shaped, dry dock bottom required by the original contract. The general dimensions for the base were: over-all depth, 15.5 ft.; bottom, 8 in. thick; sides, 8 in. thick at bottom, increasing uniformly to about 14 in. at top; ends, 10 in. thick; wales at ends and sides, about 22 in. wide and 18 in. deep; solid bulkheads, 14 in. thick, about 9.7 ft. from center to center, transverse to the dock; solid bulkheads, 10 in. thick, about 21 ft. from center to center, longitudinal to the dock; an intermediate triangular buttress between each two longitudinal bulkheads to stiffen and support the sides; all corners filleted. The base, after launching, would be fitted with timber walls and bracing to form a coffer-dam having sufficient height to reach above

tide level when the base was sunk to rest on the prepared bottom of the dock excavation. On the completion of the coffer-dam, the boat would be towed to place and gradually sunk to its final position as concrete was added to fill the bottom pockets and form the side-walls.

It was proposed to use very lean concrete, 1:3.5:7 mix, for filling the pockets in the boat and 1:3:6 concrete for the side-walls; this involved dependence on the impermeability of the thin walls and the bottom of the boat to exclude seepage and prevent the possible chemical disintegration of the lean concrete in the dock bottom, but made no provision for the safeguard of the comparatively lean concrete in the side-walls. Any unequal settlement of the ten sections, which would render the finished dock unsightly, would be corrected by placing a 6-in. layer of concrete over the entire dock floor shortly before the completion of the work, and making the total thickness of the floor 16 ft. A comparatively small quantity of transverse steel reinforcement would be embedded near the upper surface of the dock floor, to care for certain structural stresses. The spaces between adjacent sections would be sealed in the bottom by a small quantity of tremie-placed concrete which would tightly wedge between the slightly upward converging faces of the bases; concrete placed through a tremie or by other special method would be used for making the vertical seals between the side-walls.

Great care would undoubtedly be required in the execution of this plan, to prevent launching stresses, which would cause cracks or breaks in the thin walls and bottom of the bases, and to protect the sections against injury while in a floating condition, particularly when heavily loaded just before settling into position. Great care would also be necessary to have the bottom of the dock excavation perfectly prepared before placing the sections and to prevent earth from the side banks or other foreign matter from being deposited on the prepared bottom before settling a base. Any error in locating a section or serious accident to a partly completed section would be extremely costly and difficult to repair, as it would be impossible to raise a section after it had firmly settled on the bottom.

Noble Plans.—During the summer of 1913 the Secretary of the Navy retained the late Alfred Noble, Past-President, Am. Soc. C. E., as Consulting Engineer on Pearl Harbor Dry Dock matters. Mr. Noble

visited the site of the dock during August to make such personal inspections and investigations as he might desire, to supplement the voluminous records and data which had been furnished him, and on October 20th, 1913, submitted a comprehensive detailed report with his recommendations. The report emphasized the desirability of eliminating as far as possible all features of design and of construction which were experimental or hazardous, or which involved uncertainties as to the result, and urged that cost be subordinated to safety.

Mr. Noble suggested two general methods for continuing the work, which he designated as the "Type A" and "Type B" methods, either of which he thought might be advantageously adopted, but expressed a decided preference for "Type A". Both methods propose sections each about 100 ft. long, in the direction of the axis of the dock, and founded on a reinforced concrete base, built above the surface of the water and then floated and sunk in the desired position. The use of a floating dock in which to construct the bases was strongly recommended in preference to launching ways, in order to eliminate as far as possible any injury to the structures during the process of floating them, and to permit of correcting any defects in construction which might appear after immersion but before the actual removal of the bases from the floating dock.

The five-plane, dish-shaped form required by the original contract for the contour of the bottom surface of the dock prism is retained in both plans proposed by Mr. Noble, but the thickness of the dock floor at the center is increased from 16 to 18.5 ft., to eliminate or reduce the need of reinforcing metal for tensile stresses. The increase in the thickness of the floor would correspondingly increase the required excavation; that is a matter of moment, considering the fact that the dredging work would have to be conducted at such great depth below the water surface and that closely spaced piles have been driven over about four-fifths of the dock area, which would greatly complicate the difficulties attending dredging operations. It would probably be necessary to remove interfering pile heads by blasting, inasmuch as the points of cut-off are so deeply buried in the ground; the points of cut-off would necessarily be several inches below the finished bottom grade level to permit of the accurate surfacing of the bottom to receive the concrete bases. The final accurate grading of the bottom to the rather complicated form required for the support of the bases would constitute

one of the principal difficulties in the execution of these plans. Only a narrow space, 2 or 3 in. wide, would be left between adjacent sections, and the sealing of these spaces would involve little difficulty because of their narrow dimension. The very strong and stiff construction of the bases would enable them to be adjusted to considerable irregularities in the supporting value of the ground, without endangering their integrity.

The boat or base proposed for the "Type A" method would have a concrete bottom 2 ft. thick, walls 1.5 ft. thick, and 12-in. solid interior longitudinal and transverse bulkheads, about 14 ft. from center to center. All concrete would be heavily reinforced. The upper part of each base would be designed to receive a timber coffer-dam at each end, within which the side-walls for the dock would be constructed; provision would also be made for temporary sheeting and cribbing, which would be placed on top of the base between the end coffer-dam for use during the transporting and sinking of the base to position. A distinctive feature of this plan, and one which Mr. Noble considered very important, was the requirement that any base after leaving the floating dock should be quickly transported and sunk to position, thus reducing to a minimum the period during which a base was afloat and safeguarding that part of the work against accidents which might be very serious in their results. The rapid sinking of a base would be obtained by the admission of water into the coffer-dam. The open pockets formed by the interior bulkheads in the base would be filled with tremie-placed concrete, and the side-walls would be made of concrete placed under open and dry conditions within coffer-dams.

The essential difference between the "Type B" plan and that described for "Type A" is the method proposed for sinking the bases. In "Type B" the coffer-dam for each section would cover the entire area of the boat, and the boat would be sunk by the weight of the concrete filling, all of which would be placed under dry and open conditions. This plan is very similar to that suggested by Mr. Harris. Mr. Noble considered that the long period during which a base would be maintained in a floating condition while being filled with concrete introduced a building hazard of such serious nature as to render the adoption of the "Type B" method inadvisable. The ability to place nearly all concrete under favorable dry and open conditions, and the elimination from the boats of a considerable part of the longitudinal

reinforcing metal, which would have little value in the completed structure, are the principal advantages which were claimed for the "Type B" as compared with the "Type A" method.

Plate XXII shows three progressive stages in the sinking of a boat and the building of a section of the dock, as proposed under both the "Type A" and the "Type B" methods.

Hollyday Plan.—On April 21st, 1914, R. C. Hollyday, M. Am. Soc. C. E., Civil Engineer, U. S. N., recommended the sectional boat method for the construction of the dock, in case it should be decided to depart from the original contract plans. Mr. Hollyday submitted outline details based on making each section or unit 50 ft. long, measured on the axis of the dock. The boat for each section would have a bottom of two layers of 12-in. square timbers bolted together, and fitted with detachable timber ends and side-walls; the ends and sides, designed for repeated use, would be extended upward to form the walls of a coffer-dam within which all concrete would be placed in the dry. The bottom of each section would be flat. The concrete floor of the dock would be 16 ft. thick, with steel reinforcement sufficient for all tensile stresses. The boat would be sunk to position as it was filled with concrete. The plan proposed to add considerable material to the concrete side-walls of the dock and to provide horizontal projecting shelves, 2 ft. wide, outside each side-wall at the elevation of the finished floor.

The Writer's Plans.—The addition of pig iron or metal scrap to concrete to increase the weight of the mass was recommended in July, 1913, to obviate the necessity for general changes in the plans in order to make the weight of the dock greater than any possible hydrostatic uplift and to hold the center line of pressures within the middle third of the concrete mass.

The substitution of steel plates and structural shapes for reinforced concrete was suggested in September, 1913, for use in making the boats or bases required for the sectional boat method of construction.

The method of construction and general plans subsequently adopted were submitted in April, 1914. These are described in detail later under the heading "Revised Plans".

Floating Type.—The desirability of providing a dry dock of the floating type, instead of the graving type, has been suggested. It is undoubtedly true that a dock of the floating type, having ample capacity, could be constructed in less time and at less cost than one of

the graving type; this statement is especially true for a dock at Pearl Harbor where the natural soil conditions render the deep excavation and sub-surface concrete work required for a graving dock both hazardous and very expensive. However, all questions as to the type of dock to be provided are in this case beyond discussion, inasmuch as the Congressional authorization for the work distinctly directs the construction of "one graving dry dock".

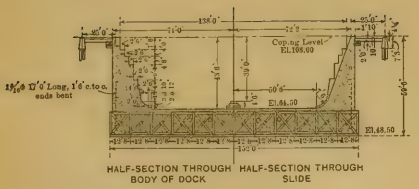
Revised Plans.

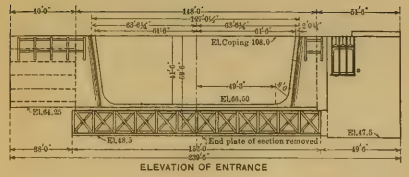
Revised plans for the dock, as shown in outline on Plate XXIII, were approved on November 19th, 1914, and its construction is now being energetically pushed in an effort to complete the work on or before June 30th, 1918.

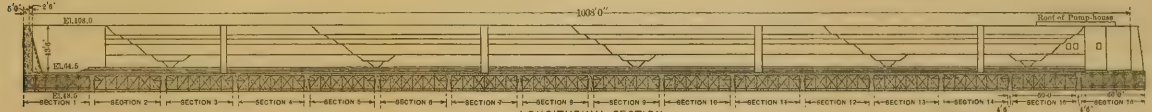
General Details.—The dock will be built in sixteen sections, each covering the entire width and having a length in the direction of the dock axis of about 60 ft. The bottom of each section will be flat, and will consist of a structural steel frame embedded in concrete. The side-walls will be of concrete with a small quantity of vertical reinforcement near their outer faces. All concrete in each section will be placed under dry and open conditions favorable for performing concrete work, which will permit of continuous and thorough inspection. There will be a space between the bases of adjacent sections, diminishing from about 5 ft. wide at the bottom to 3 ft. wide at the top, which will be sealed in the bottom with concrete placed through a tremie; the upper part of the space between the bases will be filled with concrete placed in the dry after the dock is unwatered; the considerable distance between the bases is provided to facilitate the work of a diver in clearing the space before depositing the tremie concrete. The spaces between the side-walls of adjacent sections will be sealed with small coffer-dams, and the closure will be made with concrete placed under dry conditions after unwatering the dock. Reinforcing rods will be used to tie together the bases of the two outer sections.

The semi-circular wall at the inner end of the dock will be made of large, pre-moulded, reinforced concrete, voussoir blocks in combination with concrete filling placed under dry and open conditions; a base similar in all respects to that used for the sections in the body of the dock will be used as a foundation for the end-wall; an accurately formed bed or seat will be prepared on the upper part of the base for the sup-

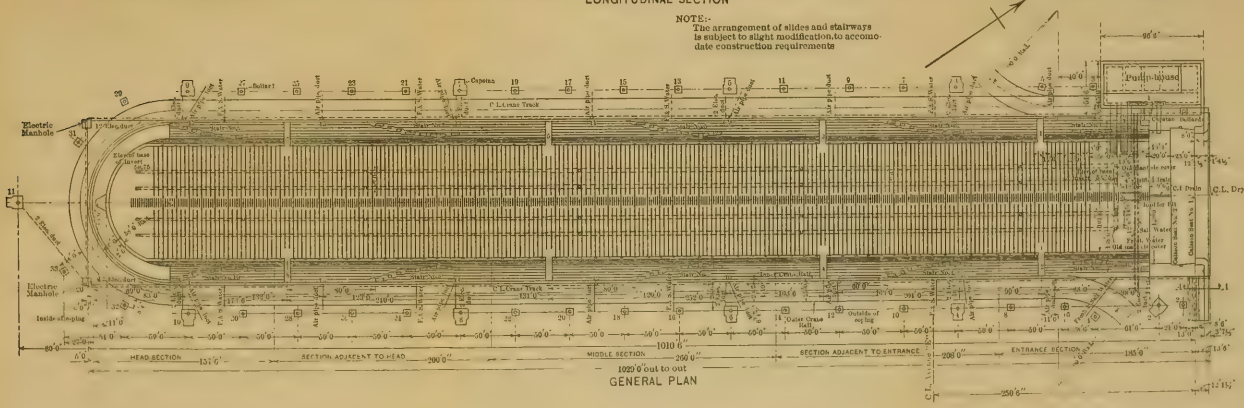
GENERAL REVISED OUTLINE PLAN OF THE DOCK







NOTE:-
The arrangement of slides and stairways is subject to slight modification, to accomodate construction requirements



port of the wall; the voussoir blocks will weigh about 150 tons each, and will be made in forms on the bank adjacent to the excavation, within reach of the Station's large floating crane, which will be used in setting the blocks; a space about 4 ft. wide will be left between adjacent voussoirs, and this will be closed by an inner and an outer coffer-dam, after which the space will be unwatered and filled with concrete; a keyway or grouting space will be provided between the bottom of each block and its bed, which will be sealed when the concrete is placed in the open space adjoining the block.

Additional concrete will be placed under the side-walls, to make the bottom of the bases flat, and additional piles will be driven to support this concrete. The pump-well will be near the entrance, and will be integral with the side-wall of the dock. The pump-well will be founded on a steel boat supporting a timber coffer-dam; the boat will be sunk by the addition of the concrete required in the construction of the pump-well. The concrete will generally be made much richer than originally required.

Stress Analyses.—The structural details are based on numerous stress computations, which in turn are based on various assumptions, as follows:

That maximum tide will reach an elevation of 102.5, which is 2.5 ft. above mean low tide and 1.3 ft. above mean high tide.

That sea water weighs 64 lb. per cu. ft.

That effective hydrostatic pressure on the bottom of the dock will equal full sea pressure at that depth, due to the extremely porous character of the soil.

That the earth side-banks of the excavation for the dock will take a natural slope of 1:1 both above and below tide level.

That the soil above extreme high tide level at Elevation 102.5 weighs 102 lb. per cu. ft., and that its submerged weight is 63 lb. per cu. ft. below this level. The equivalent liquid pressures on a vertical surface are 17.5 and 10.8 lb. per sq. ft., respectively.

That concrete weighs 150 lb. per cu. ft.

That there are transverse steel trusses embedded in the concrete bottom of the dock which provide the equivalent, per linear foot of dock, of 3 sq. in. of metal in the top chord and 1.6 sq. in. in the bottom chord.

That there is water pressure, but no earth pressure, on those portions of the upper horizontal surface of the base which project beyond the outer vertical faces of the side-walls.

That there is no Artesian pressure which would affect the hydrostatic pressure on the bottom of the dock.

On the foregoing assumptions, the dock was analyzed to determine the stresses which would result within its structure, depending on varying external forces and the manipulation of the dock. The stress diagrams on Plates XXIV and XXV have been made for the following cases:

Case A.—Uniform soil and water pressure under the bottom of the dock; dock empty; external water and earth pressure on side-walls.

Case B.—Uniform soil and water pressure under the bottom of the dock; dock empty; external water pressure only on side-walls.

Case C.—Uniform soil pressure under the bottom of the dock; dock full of water; external earth pressure on side-walls.

Case D.—Uniform soil pressure under the bottom of the dock; dock full of water; external water pressure only on side-walls.

Case E.—Uniform soil and water pressure under the bottom of the dock; ship load of 200 000 lb. per lin. ft. of dock, 120 000 lb. on keel block, and 40 000 lb. on each side-block; external water and earth pressure on side-walls.

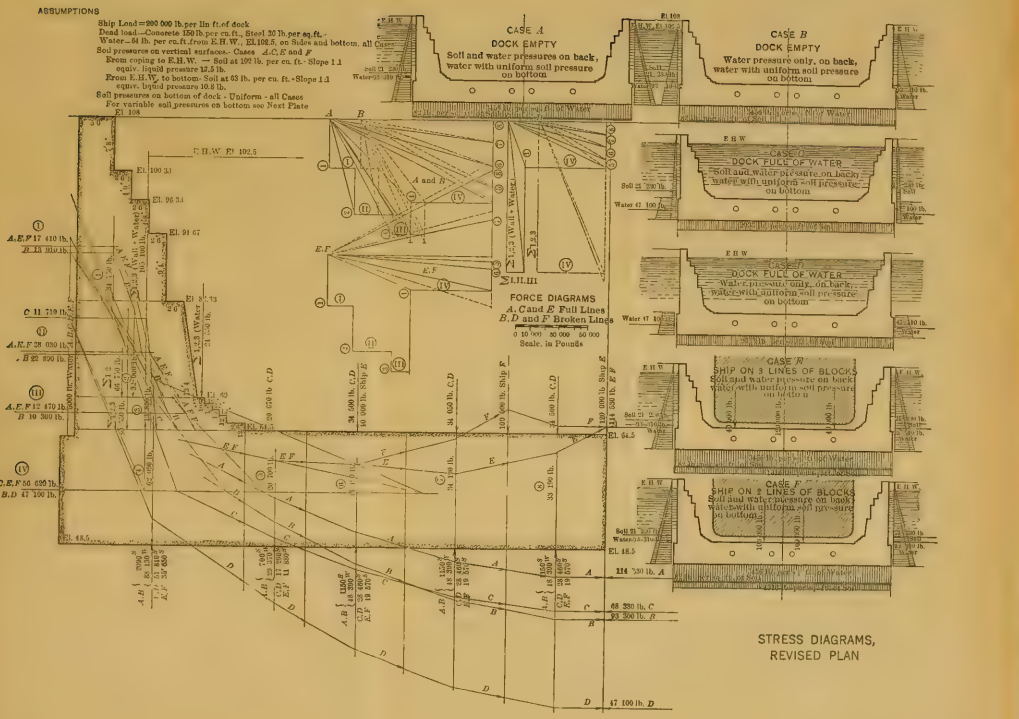
Case F.—Uniform soil and water pressure under the bottom of the dock; ship load of 200 000 lb. per lin. ft. of dock, one-half on each side-block; external water and earth pressure on side-walls.

Case G.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock empty; external water and earth pressure on side-walls.

Case H.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock empty; external water pressure only on side-walls.

Case I.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock full of water; external water and earth pressure on side-walls.

Case J.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock full of water; external water pressure only on side-walls.


ASSUMPTIONS
Ship Load = 200 000 lb. per lin ft. of dock
Dead load—Concrete 150 lb. per cu. ft., Steel 30 lb. per sq. ft.
Water—64 lb. per cu. ft. from E.H.W., El. 102.5, on Sides and bottom, all Cases
Soil pressure on vertical surfaces.- Cases A, C, E and F
From coping to E.H.W. — Soil at 102 lb. per cu. ft. - Slope 1 1
equiv. liquid pressure 12.5 lb.
From E.H.W. to bottom- Soil at 63 lb. per cu. ft. - Slope 1 1
equiv. liquid pressure 10.8 lb.
Soil pressures on bottom of dock - Uniform - all Cases
For variable soil pressures on bottom see Next Plate
El. 108
CASE A
DOCK EMPTY
Soil and water pressures on back,
water with uniform soil pressure
on bottom
CASE B
DOCK EMPTY
Water pressure only, on back,
water with uniform soil pressure
on bottom
CASE C
DOCK FULL OF WATER
Soil and water pressure on back,
water with uniform soil pressure
on bottom
CASE D
DOCK FULL OF WATER
Water pressure only, on back,
water with uniform soil pressure
on bottom
CASE E
SHIP ON 3 LINES OF BLOCKS
Soil and water pressure on back,
water with uniform soil pressure
on bottom
CASE F
SHIP ON 2 LINES OF BLOCKS
Soil and water pressure on back,
water with uniform soil pressure
on bottom
FORCE DIAGRAMS
A, C and E Full Lines
B, D and F Broken Lines
Scale, in Pounds
E.H.W. El. 102.5
El. 100.33
El. 96.34
El. 91.67
El. 87.33
El. 82
El. 54.5
El. 48.5
A and B
E F
I, II, III
I A.E.F 17 410 lb.
B 13 910 lb.
II A.E.F 58 050 lb.
B 25 890 lb.
III A.E.F 12 470 lb.
B 10 360 lb.
IV A,C,E,F 56 620 lb.
B,D 47 100 lb.
34 000 lb. C,D
10 000 lb. Ship E
34 400 lb. C,D
10 000 lb. Ship E
34 100 lb. C,D
10 000 lb. Ship E
30 150 lb. E F
30 670 lb. C,D
30 200 lb. E
34 190 lb. E
30 150 lb. E
114 530 lb. A
68 280 lb. C
53 200 lb. B
47 100 lb. D

STRESS DIAGRAMS,
REVISED PLAN

Case K.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; ship load of 200 000 lb. per lin. ft. of dock, 120 000 lb. on keel block and 40 000 lb. on each side-block; water and earth pressure on side-walls.

Case L.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; ship load of 200 000 lb. per lin. ft. of dock, one-half on each side-block; external water and earth pressure on side-walls.

The load per square foot on the soil underlying the dock is: for Case *A*, 82 lb.; Case *B*, 82 lb.; Case *C*, 2 033 lb.; Case *D*, 2 033 lb.; Case *E*, 1 398 lb.; Case *F*, 1 398 lb.; Case *G*, 82 lb. under the sides, increasing to 217 lb. at a distance of 50.5 ft. from the center, then decreasing to zero at a point 28 ft. from the center; Case *H*, same as Case *G*; Case *I*, 2 350 lb. under the sides and to a point 50.5 ft. from the center, then uniformly decreasing to 1 395 lb. at the center; Case *J*, same as Case *I*; Case *K*, 82 lb. at the sides, increasing uniformly to 2 007 lb. at a point 28 ft. from the center, then uniform to the center; Case *L*, 82 lb. under the sides, increasing uniformly to 2 304 lb. at a point 14 ft. from the center, then uniform to the center.

The pressure on the soil is so small when the dock is empty, as treated in Cases *A* and *B*—only 82 lb. per sq. ft.—that it seems improbable that there will be any great variation in the intensity of the pressure. Assuming, however, that there will be a variation, as given under Cases *G* and *H*, there would be a reduction in the bending moment at the center of the dock, which, in comparing Case *G* with Case *A*, would reduce the stresses in the steel and concrete from 15 430 and 472 lb. per sq. in., respectively, to 12 150 and 412 lb.; and, comparing Case *H* with Case *B*, would reduce the unit stress from 25 330 and 592 lb. to 23 260 and 558 lb. These latter stresses, as computed for Cases *B* and *H*, cannot actually occur, as the specification governing the work requires that the back-filling outside the side-walls of the dock shall be placed before the dock is emptied.

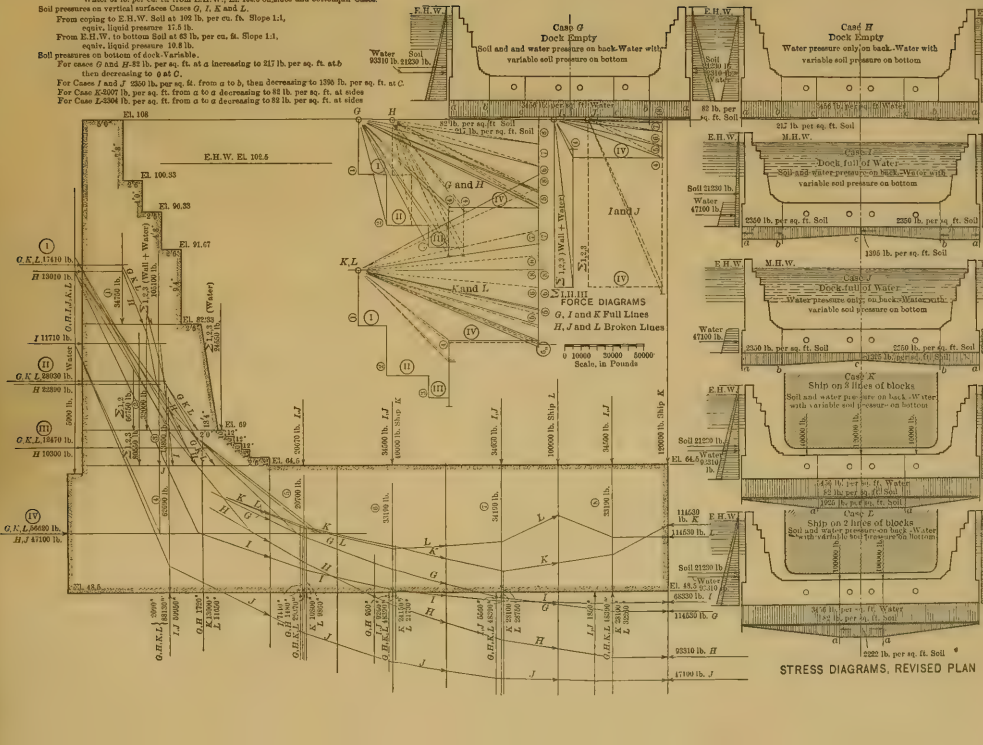
The slight variation in soil pressure beneath the dock, computed for Cases *I* and *J*, as compared with the condition of uniform pressure computed for Cases *C* and *D*, results in reducing the bending moment at the center of the dock. Comparing Case *I* with Case *C*: this reduces the unit stresses in the steel and concrete from 17 300 and 413 lb. to 5 480 and 213 lb. Comparing Case *J* with Case *D*: the unit stresses

in the steel and concrete are reduced from 28 350 and 535 lb. to 15 080 and 334 lb. The unit stresses would be still further reduced with further small variations in the intensity of the supporting soil pressure. Inasmuch as a considerable deformation or strain of the dry dock structure would necessarily have to occur before destructive unit stresses could be developed, and as these changes would assuredly introduce variations in the intensity of the soil pressure, with reductions in stresses even greater than those previously computed, it may be concluded that the dock as designed will be safe when filled with water.

Slight variations in the intensity of the soil pressure beneath the dock, to conform with the variations in the superimposed load when the dock is pumped out and carries a ship on the blocks, result in comparatively large changes in the unit stresses in the dock structure. Assuming variations in the soil pressures, which would logically follow from the more or less elastic nature of the dock structure when under strain, and with variations confined within the narrow limits computed for Cases *K* and *L*, as compared with the uniform pressures computed for Cases *E* and *F*, shows that the unit stress in the steel in the bottom chord would be reduced from 14 140 lb. tension in Case *E* to 750 lb. compression for Case *K*, with corresponding reduction in the unit compression in the concrete from 386 to 50 lb.; comparing Case *L* with Case *F*, the unit stress in the bottom chord steel is changed from 14 140 lb. tension to 116 lb. compression, and the unit compression in the concrete is reduced from 386 to 80 lb. From these figures it is concluded that the dock can safely carry the assumed ship loading.

It is apparent from the foregoing analysis that the critical condition of loading is that shown by the stress diagram, *A*, which assumes the dock to be empty, with hydrostatic and earth pressures on the side-walls. Slight variations in the intensity of the soil pressure beneath the dock, for this condition, will have very little effect on the unit stresses in the dock structure. The center line of pressure within the bottom of the dock prism may be moved farther into the body of concrete by assuming an induced passive soil pressure or greater active soil pressure on the side-walls, but this would bring the center line of pressure dangerously near the haunch or toe of the side-walls; vertical reinforcement in each side-wall, equivalent to 0.9 sq. in. of steel per lin. ft. of dock, has been provided to meet this contingency.

ASSUMPTIONS:- Ship load—220 000 lb. per lin. ft. of Dock
Dead load—Concrete 150 lb. per cu. ft., Steel 30 lb. per sq. ft. bottom.
Water 64 lb. per cu. ft. from E.H.W., El. 102.5 on sides and bottom, all Cases.
Soil pressures on vertical surfaces Cases *G*, *I*, *K* and *L*.
From coping to E.H.W. Soil at 102 lb. per cu. ft. Slope 1:1, equiv. liquid pressure 17.5 lb.
From E.H.W. to bottom Soil at 63 lb. per cu. ft. Slope 1:1, equiv. liquid pressure 10.8 lb.
Soil pressures on bottom of dock-Variable.
For cases *G* and *H*-82 lb. per sq. ft. at *a* increasing to 217 lb. per sq. ft. at *b* then decreasing to 0 at *C*.
For Cases *I* and *J* 2260 lb. per sq. ft. from *a* to *b*, then decreasing to 1390 lb. per sq. ft. at *C*.
For Case *K*-6007 lb. per sq. ft. from *a* to *a* decreasing to 82 lb. per sq. ft. at sides
For Case *L*-5304 lb. per sq. ft. from *a* to *a* decreasing to 82 lb. per sq. ft. at sides


Case G
Dock Empty
Soil and and water pressure on back-Water with variable soil pressure on bottom
Case H
Dock Empty
Water pressure only on back-Water with variable soil pressure on bottom
Case I
Dock full of Water
Soil and water pressure on back-Water with variable soil pressure on bottom
Case J
Dock full of Water
Water pressure only on back-Water with variable soil pressure on bottom
Case K
Ship on 3 lines of blocks
Soil and water pressure on back-Water with variable soil pressure on bottom
Case L
Ship on 2 lines of blocks
Soil and water pressure on back-Water with variable soil pressure on bottom
FORCE DIAGRAMS
G, I and K Full Lines
H, J and L Broken Lines
Scale, in Pounds
STRESS DIAGRAMS, REVISED PLAN

The weight of the dock is 12 480 lb. per lin. ft. of dock in excess of the hydrostatic pressure on the bottom.

The safety and sufficiency of the dock structure are further insured by various forces which have not been evaluated in the foregoing treatment, but which have material magnitude and will be positive and reliable in their action. These additional elements of strength are in the nature of factors of safety, and include the following items:

(*a*).—The pile-supported concrete prism, which surrounds the dock to carry the outer rail of the crane track, is secured to the side-walls of the dock near the coping level by closely placed reinforced concrete ties, and serves to anchor or support the side-walls against inward movement.

(*b*).—The "plums" or "one-man stone", which will be embedded in the concrete, weigh from 167 to 173 lb. per cu. ft. The unit weight of 150 lb. assumed for concrete in making the stress diagrams was not increased on account of these "plums". The specifications provide that as large a percentage of plums may be used as may be consistent with the obtaining of strong, dense, and impervious concrete. The addition of plums will undoubtedly add quite a little to the assumed average unit weight of the mass concrete.

(*c*).—It has just been shown that the critical loading condition for the dock is that discussed under Case *A*, which assumes the dock to be empty. Under this loading condition, the weight of the blocking equipment which will be secured to the floor of the dock will be directly effective, and will reduce materially the unit stresses in the dock floor.

(*d*).—The weight of the soil on the shelves which project beyond the outer faces of the side-walls will add to the weight of the dock to resist flotation in the structure.

Special Plant.—The contractors are providing two special items of plant which are necessary for carrying on the work. These consist of a floating dry dock and a steel structure which, for want of a better name, will be called a coffer-dam-boat.

The floating dock is being made of timber, and will have a lifting capacity of about 3 500 tons. The general dimensions of the dock are: over-all width, 100 ft.; clear width between side-walls, 76 ft.; depth of bottom pontoons below floor, 12 ft.; over-all length, 214 ft.; height of side-walls above floor of dock, 34 ft.; top width of side-walls, 8 ft.;

length of side-walls, 164 ft. The bottom is divided into five pontoons, each 32 ft. wide, with a space 12 in. wide between adjacent pontoons. There is an outrigger, 25 ft. long, at each end of the dock. Each pontoon has a small side-wall, 4 ft. high and 12 ft. wide, at each end, to which the main side-walls of the dock are secured. The maximum draft of the floating dock is 41.5 ft., and will occur when the dock is lowered to receive the coffer-dam-boat; the top of the floating dock will be 4.5 ft. above the water surface in this position. There is a bracket on the inner face and near the top of each side-wall which will be loaded with about 100 tons of ballast to relieve the bending moment in the pontoons under certain conditions of loading.

The coffer-dam-boat is a structure which fulfills several requirements; it is a true coffer-dam, as it will exclude water from a working space which is open to the atmosphere and is below sea level; it is a boat, because it will be used for transporting and for containing a load, and because it can be alternately sunk and floated. Its service as a boat will be actually required to transport the partly completed bases for the sections of the concrete dock and to contain water which will be used to increase its weight; it will be repeatedly sunk, to locate the bases on the bottom and to impose its loaded weight as ballast on the bases to resist hydrostatic uplift; it will be repeatedly floated, to clear the completed dock sections of falsework and for repeated use in handling the bases on which the sections are founded. The structure will be entirely of steel, with the exception of a small quantity of removable timber bracing in the end pontoons, and may be described as an open boat, 108 ft. long, 60 ft. wide, and 38.5 ft. deep, with sides extended longitudinally and downward to form a U-shaped coffer-dam which will surround the ends and bottom of the boat. The coffer-dam space at each end of the boat is 22 ft. wide, and is 6 ft. $3\frac{5}{8}$ in. high under the boat. There is permanent steel bracing, together with removable timber bracing, in each vertical leg of the coffer-dam. Numerous braced steel stools are connected to the bottom of the boat, for use while transporting a base and to support the weight on the bottom when the boat is sunk and resting on a base frame. The interior space in the boat is sub-divided into four equal water-tight compartments by a transverse and a longitudinal bulkhead; these compartments are fitted with pumps, piping, and valves for use in adjusting the trim of the boat; the bulkheads are not heavily braced or stiffened, as only a

small difference in level of the water in the four compartments is anticipated. Figs. 13 and 14 are photographs of a model showing the general form and construction of the coffer-dam-boat and giving some idea of the heavy framing and bracing required. The estimated weight of the structure is 875 tons.

Field Work.—A sufficient area underlying the dock will first be cleared of all earth, débris, and pile heads to a level sub-grade 12 in. below the finished grade. Shortly before placing the base for any section of the dock, the area which is to be covered will be swept by a frame having its lower edge held at a level 6 in. below the finished grade, to be sure that all projecting points have been removed, after which the area will be carefully brought to the required grade with broken stone and sand filling.

All piles which have been driven and were injured by the previous upheaval, or which will be too short to be reliable after the excavation is deepened, will be replaced with suitable piling. The additional piling required for the supports of the heavier dock will also be driven.

The structural steel frame required for the base of each section will weigh about 153 tons. This steelwork is being fabricated at Ambridge, Pa., in accordance with the standard specifications for material and workmanship contained in the 1914 handbook of the United States Steel Products Company for steelwork for buildings, using steel covered by Classes "B" and "C". Fig. 10 is a photograph of a model of the steel frame for one of the bases, and Fig. 11 shows the base as it will appear when erected on the floor of a floating dry dock. It may be noted that the model of the floating dock shown on Fig. 11 and on any subsequent illustrations is that of an assumed dock, and does not represent the one now under construction and previously described.

The steel frame comprises seven trusses, 152 ft. long, and 10 ft. from center to center; the trusses will be in a direction transverse to the axis of the dock when in position. Each truss has twelve panels and is 14.75 ft. deep over all. Adjacent trusses will be connected by cross-bracing at each panel point. The top chords of the interior trusses in the frame at the center panels are composed of two 6 by 6 by $\frac{5}{8}$-in. angles, one 20 by $\frac{1}{2}$-in. plate, and two $14\frac{1}{2}$ by $\frac{3}{8}$-in. side-plates; the lower chords of the interior trusses are composed of two 6 by 4 by $\frac{9}{16}$-in. angles and one $14\frac{1}{2}$ by $\frac{3}{8}$-in. plate. The intermediate posts in the interior trusses are composed of four 6 by 4 by $\frac{9}{16}$-in. angles and one

12 by $\frac{1}{2}$-in. plate, and are very heavy, in order to carry the water load of the boat when the base is first settled into position. The exterior trusses of each frame and the end bracing connecting the seven trusses in a frame have no diagonal web members; the top chords are made of a 15-in. 33-lb. channel, with the web horizontal, in combination with a vertical 30 by $\frac{5}{16}$-in. web-plate, which is stiffened near the lower edge with a 3 by 3 by $\frac{5}{16}$-in. angle; the angles in the lower chords have their vertical legs turned down, in order that the outstanding legs will be more effectively protected by the concrete matrix in which the frame is to be embedded. The steel frames will be erected, aligned, and riveted on the floor of the floating dock.

The concrete work may be separated into two principal classes; the first will include practically all concrete which is placed under dry and favorable conditions, and the second will include that placed under water through a tremie. About 133 700 cu. yd. of concrete will be placed under dry conditions, including the massive work in the body of the dock, in the pump-well, and in the portions of the quay walls adjacent to the dock entrance; this concrete will be mixed in the proportion of 100 lb. of cement, $\frac{3}{4}$ cu. ft. of coral sand, $1\frac{1}{2}$ cu. ft. of crusher screenings, and 4 cu. ft. of broken stone; "plums" or large pieces of rock may be embedded in the concrete in quantity which will be consistent with obtaining satisfactorily dense, strong, and impervious concrete. About 6 500 cu. yd. of concrete will be placed under water through a tremie, which includes about 2 500 cu. yd. for making the seals between the bases of adjacent sections and under the voussoirs at the head of the dock and 4 000 cu. yd. in the pump-well; this concrete will be mixed in the proportion of 100 lb. of cement, $\frac{2}{3}$ cu. ft. of coast sand, $1\frac{1}{3}$ cu. ft. of crusher screenings, and $3\frac{1}{2}$ cu. ft. of broken stone. The exposed faces of the side-walls and the wearing surface of the floor of the dock, also the interior walls and floors of the pump-well and connected culverts, will be finished with a coating of "lavalithic", which will be placed at the same time as the concrete backing; the "lavalithic" will be mixed in the proportion of 100 lb. of cement, $\frac{1}{2}$ cu. ft. of coast sand, and $1\frac{1}{2}$ cu. ft. of crusher screenings; and the coating will be from $1\frac{1}{2}$ to 2 in. thick.

Work will be started placing the concrete to embed a steel base frame just as soon as the erection of the steel frame is completed. A layer of thin boards or other suitable material will be placed on the

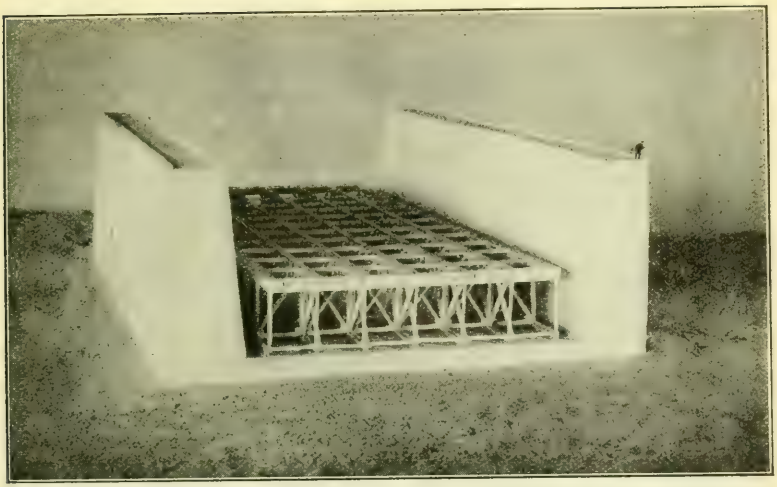


FIG. 11.—STEEL FRAME FOR BASE OF SECTION ERECTED ON FLOATING DOCK.

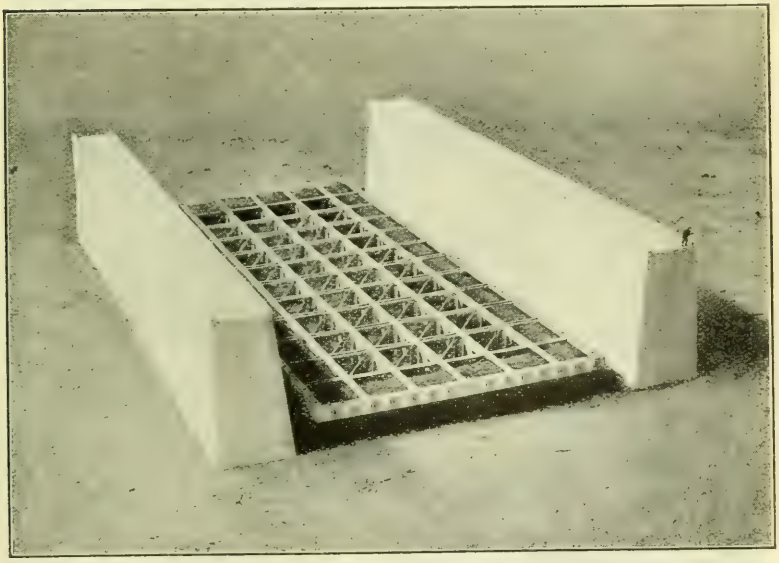


FIG. 12.—STEEL FRAME WITH EMBEDDING CONCRETE PARTLY PLACED.
(At this stage the partly completed base is ready to be removed from the floating dock.)

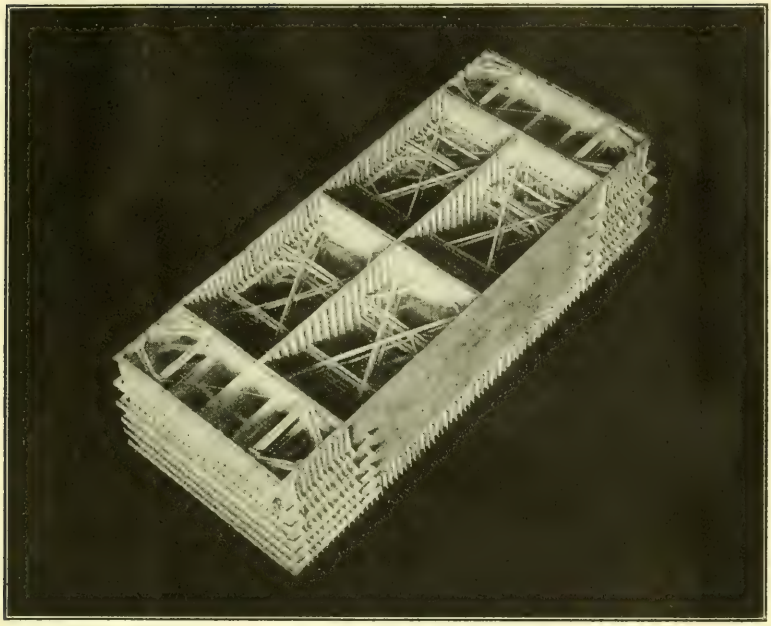


FIG. 13.—TOP VIEW OF COFFER-DAM-BOAT USED FOR TRANSPORTING AND SINKING SECTION BASES, AND REQUIRED FOR THE COMPLETION OF EACH SECTION.

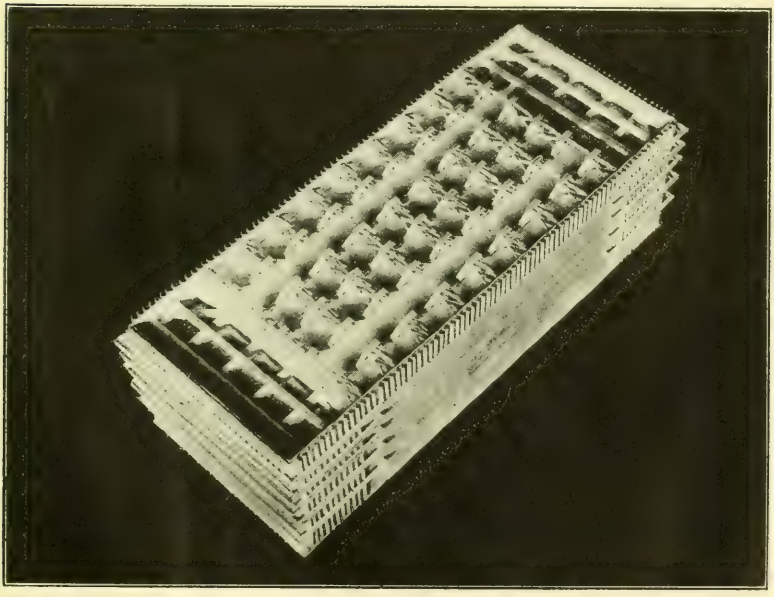


FIG. 14.—BOTTOM VIEW OF COFFER-DAM-BOAT USED FOR TRANSPORTING AND SINKING SECTION BASES, AND REQUIRED FOR THE COMPLETION OF EACH SECTION.

floor of the floating dock before the steel frame is erected and on which the concrete will be placed; this loose covering will protect the dock floor from injury when the concrete is placed, and will tend to prevent the suction or adhesion between the base and the floor of the floating dock when the base is removed. The steel frame and side-walls of the floating dock will provide ready means for securing and shoring the form work required to confine the concrete. About 3 500 cu. yd. of concrete will be placed for each base while the structure is carried on the floating dock; this concrete will be about 8 ft. thick over the entire area of the base, and will be banked solidly around the side-walls and ends to the extreme top of the steel frame; care will be taken to obtain a tight seal between the banked concrete and the vertical 30-in. plate which forms part of the top chord of the side girders and end bracing of the steel frame; the thickness of the concrete banks will be made ample to prevent any inward movement of the sides and ends of the base when subjected to the pressures which will occur during the later stages of the work; the depth of 8 ft. of concrete in the bottom of the base is required for structural strength, and also to make the combined submerged weight of the base and coffer-dam-boat sufficient to resist the hydrostatic uplift when the base is settled into position and the coffer-dam unwatered. The estimated weight of the partly completed base when ready to be removed from the floating dock is about 7 250 short tons; the estimated displacement of the base, assuming the dished interior to be free of water, is about 4 320 short tons. Fig. 12 is a photograph showing the appearance of a partly embedded steel frame resting on the floating dock and ready to be removed.

After completing that portion of the work on a base which is performed while it is on the floor of the floating dock, the dock will be lowered and the coffer-dam-boat floated to position over the base. The floating dock is then raised until it has lifted the joint between the base and the superimposed coffer-dam-boat to a level above the surface of the water, when the joint will be made with bolts using a lead gasket between the top flange of the base and the lower flange of the coffer-dam to obtain water-tightness. The stools under the bottom of the boat will bear over panel points of the trusses in the steel frame when the coffer-dam-boat and base are connected and the stools and frame will be connected with bolts.

The top chords of the outside trusses and end girders of the steel frames are fitted with exterior shelf-angles forming pockets to receive the lower ends of bolts which will be used to bond together the coffer-dam-boat and bases; these bolts will be long enough to reach the top of the coffer-dam to facilitate their detachment after completing a section of the dock without requiring the services of a diver. The bolts connecting the stools combined with the exterior bolts will have ample strength to carry the weight of a suspended base. Fig. 15 shows the coffer-dam-boat in position over a base with the floating dock raised to bring the joint between the coffer-dam and the base above the surface of the water. Fig. 16 shows the floating dock lowered sufficiently to float the coffer-dam-boat with its suspended base or load which is ready to be transported to the desired location over the dock site. All the water which entered the partly completed dish-shaped base when the floating dock was lowered to receive the coffer-dam-boat will be pumped out after the joint between the coffer-dam and base is completed, and as this space is then fully protected against the entrance of water by the walls of the coffer-dam with its temporary concrete steel bottom, there should be no difficulty in keeping the space dry during the transportation and sinking of the base and while the concrete is being placed to complete the dock section of which the base will form a part.

When the coffer-dam-boat with suspended base has been floated to the desired location, the boat will be sunk by admitting water into its interior compartments until the base comes to rest on the bottom of the dock excavation. The rate of sinking and trim of the boat will be under perfect control during the entire sinking process, and, by regulating the quantity of water in the boat, the base could readily be raised and again sunk if it should be desired to clear the bottom or to obtain a better location. After the base has been satisfactorily placed, it will be firmly held in position by the weight of the coffer-dam-boat aided by the weight of the water in the boat. Fig. 17 shows the coffer-dam-boat resting over a base which has been placed; the boat is full of water but the space within the coffer-dam is dry. The concrete required to complete the section to a point above high-tide level will then be placed in the dry space within the coffer-dam; the temporary bracing in the ends of the coffer-dam will be modified as required for the construction of the side-walls. The

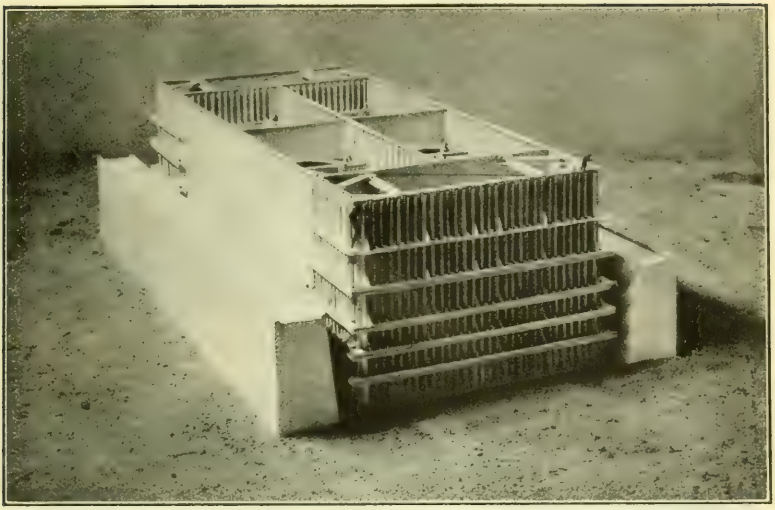


FIG. 15.—COFFER-DAM-BOAT MOUNTED ON SECTION BASE, READY FOR SINKING THE FLOATING DOCK.

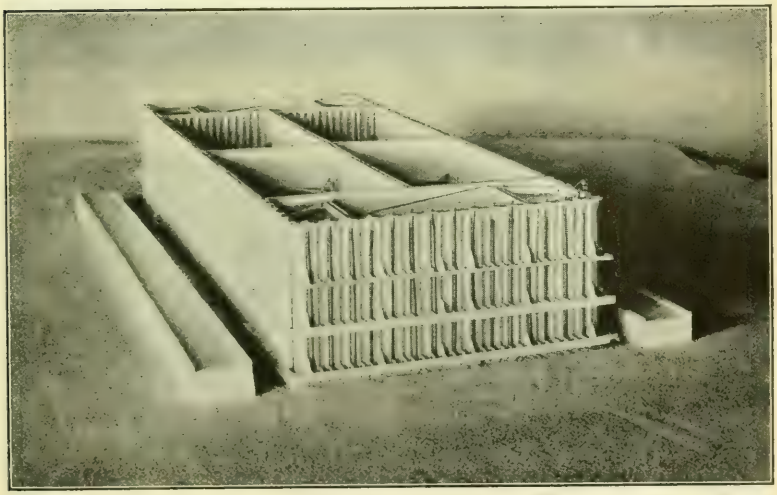


FIG. 16.—FLOATING DOCK IN DEEPEST POSITION, JUST PREVIOUS TO REMOVAL OF COFFER-DAM-BOAT WITH ITS ATTACHED BASE.

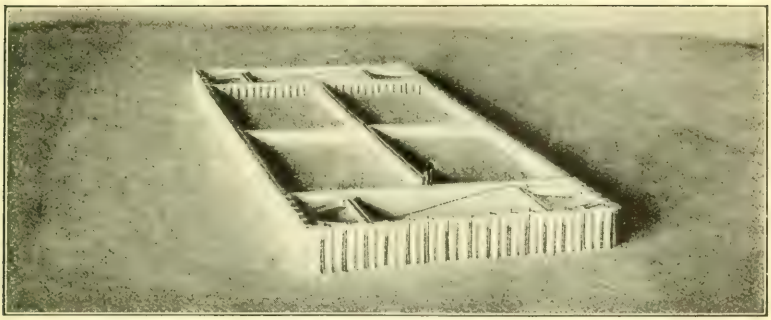


FIG. 17.—COFFER-DAM-BOAT IN FLOODED CONDITION.

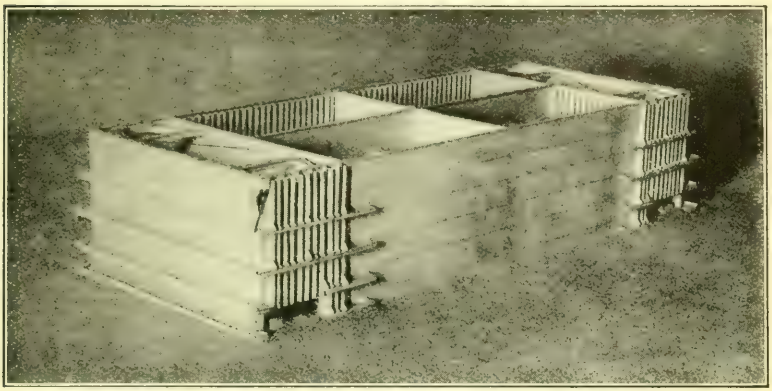


FIG. 18.—COFFER-DAM-BOAT DETACHED FROM BASE, AND GATES IN SIDES OF COFFER-DAMS OPEN TO CLEAR SIDE-WALLS OF COMPLETED DOCK SECTION.

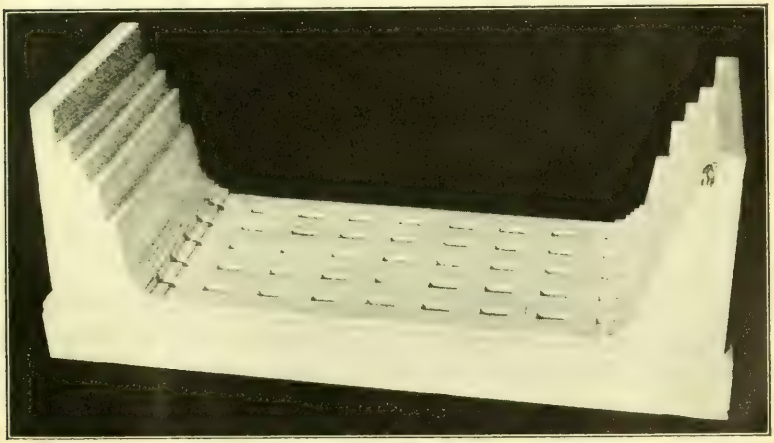


FIG. 19.—ISOMETRIC VIEW OF COMPLETED SECTION OF DOCK.

bracing between the stools under the boat will be shifted to facilitate the concrete work.

When the concrete work which must be performed within the coffer-dam is completed for any section, the bolts connecting the stools of the boat with the steel frame will be removed and the coffer-dam flooded; water will be pumped into the coffer-dam space to a level a little above that of the tide to produce an outward pressure on the walls of the coffer-dam to liberate any temporary bracing which may have been placed between the concrete and the walls of the coffer-dam; the exterior bolts will then be removed and the water pumped out of the boat; when the boat is practically dry the side-gates in the ends of the coffer-dam will be removed to enable the coffer-dam-boat to be floated over the concrete side-walls of the completed section. Fig. 18 shows the coffer-dam-boat floating at minimum draft, with the coffer-dam gates opened ready for the coffer-dam-boat to be removed from a completed section to repeat the cycle of operations.

Fig. 19 shows an isometric projection of a completed section. The rectangular spaces in the floor are the spaces in which the stools rested; they will be filled with concrete when the dock is unwatered. The concrete below the surface of the floor to the top of the steel frame is 15 in. thick.

The semicircular wall at the head of the dock will be built on a bed which will be prepared within the steel coffer-dam prior to the detachment of the coffer-dam-boat from the base for the inner section.

After completing the sixteen sections of the dock, and the pump-well, the open spaces between the sections and the well will be sealed with tremie-placed concrete or small coffer-dams and the dock unwatered. The open spaces will then be completed under dry and open conditions, and the side-walls will be carried to the coping level.

The portions of the work which have not been described are not considered to involve any special construction difficulties.

The approved design incorporates the sectional-boat method of construction, first suggested by Mr. Cox in 1911, the flat bottom, suggested by Mr. Harris, the use of water for weighting the coffer-dam, suggested by Mr. Cox, and the use of a floating dry dock for the construction of bases and the rapid sinking of the bases, suggested by Mr. Noble. Features which are believed to have originated with the writer include the use of structural steel frames in combination with concrete for making the bases, instead of reinforced concrete;

the transporting or carrying device to insure the bases against sinking or loss from the time they are released from the floating dock until they are finally settled in position in the bottom of the dock excavation; the facilities for readily adjusting the draft and trim of the bases while being moved to position and being settled; the provision which enables the repeated settling and raising and settling of any base, as may be desired, to secure accurate alignment and satisfactory bed; the elimination of all timber sheeting and nearly all timber bracing required for construction of the dock sections, with large reduction in cost and building hazard; the elimination of practically all tremie-placed concrete without involving endangering conditions; and the use of voussoir blocks in building the semicircular wall at the inner end of the dock.

The elimination of tremie concrete is considered a most valuable feature of the approved plans, inasmuch as there can be no absolute certainty that concrete deposited under water will possess the uniform degree of perfection which is essential if the dock is to be satisfactory when completed. Although other methods which have been suggested would obviate the need for tremie concrete, the results would be accomplished at the expense of serious building hazards involved by long periods of flotation while unwieldy boats of more or less temporary construction were being loaded with large masses of concrete, and by the repeated construction and use of large timber coffer-dams. The finished details of the plans represent the combined studies of the several members of the Corps of Civil Engineers who have worked on the problem, the contractors' engineers, and the Bureau's technical assistants.

Conclusion.

The writer takes pleasure in mentioning the valued assistance rendered him in the preparation of this paper by Mr. E. R. Gayler, and Mr. Joseph Michaelson. The record would not be complete, however, without special tribute to the late Alfred Noble. He should be given the credit for final success in bringing about a resumption of work under most amicable arrangements between the contractors and the Government, in accordance with plans which apparently provide the greatest practicable safeguards in the expenditure of the country's funds with every promise of the ultimate satisfactory completion of the greatest public work of the United States Navy.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE BURDEN WATER-WHEEL*

By F. R. I. Sweeny, Esq.

Synopsis.

The south supporting pier of the old Burden water-wheel at Troy, N. Y., gave way on August 22d, 1914, and since that time there has been a general and rapid decay. In view of the fact that this wheel was the greatest in America, of a type now virtually obsolete, it seems proper at this time that its history, construction, and operation should be reviewed and placed on record in the publications of this Society. The writer was fortunate in having obtained complete measurements of the dimensions of the wheel and its setting prior to its failure, and these are given in the accompanying illustrations.

A paper† by the late Joseph P. Frizell, M. Am. Soc. C. E., entitled "The Old-Time Water-Wheels of America", unfortunately, did not mention this wheel. In the discussion of that paper, however, the desire was expressed that it be pictured, "and its efficiency, present condition and relative usefulness made known". Partly in response to this and to the feeling previously expressed, this paper is presented.

Historical.

Water-wheels, in the past, have been used for a variety of purposes, but it is doubtful if there ever existed a case having such a historic setting as the old Burden water-wheel. It is intimately associated with

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited, for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

† *Transactions*, Am. Soc. C. E., April, 1893, Vol. XXVIII, p. 237.

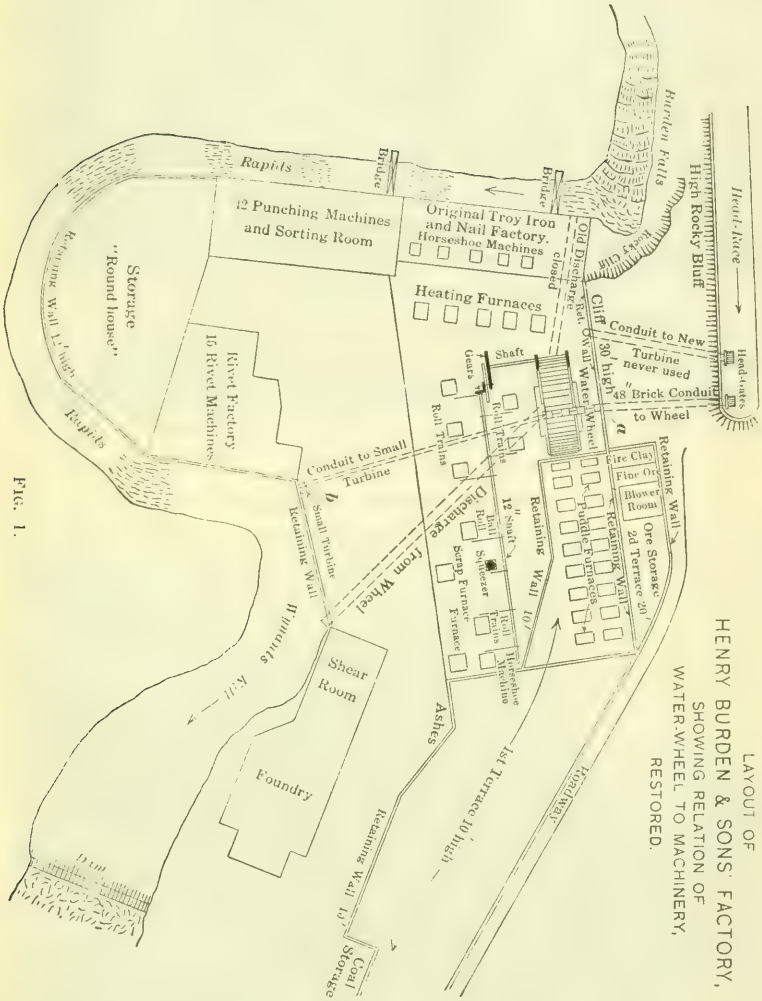
those great inventions of its designer and was born of the same genius, so that it is difficult to separate it from its setting. Thus, in order to understand the conditions leading up to its origin, it is necessary to revert to a brief historical outline prior to its inception.

Henry Burden, the designer of the wheel, was born in Dunblane, Scotland, in 1791, and after receiving an engineering education in Edinburgh, came to the United States in 1819, settling in Albany, N. Y., where he engaged in the manufacture of agricultural implements. In 1822 he moved to Troy, and took over the superintendency of the Troy Iron and Nail Factory, at that time a comparatively insignificant establishment. He began at once on his career as an inventor and manufacturer, changing the whole process of the manufacture of wrought iron and its products, and transforming the Troy Iron and Nail Factory into the largest works of the kind. It will suffice to state that it was here that he invented, built, and, for years, operated machines for the manufacture of ship and railroad spikes, nails, and horseshoes, as well as his concentric squeezer for reducing puddle balls. Thus, it will be noted that he was very progressive, constantly aiming to operate the plant more efficiently. The factory became very profitable, and because of his various inventions, Mr. Burden gradually acquired full interest, and in 1840 became proprietor. The firm was known thereafter as Henry Burden and Sons. It appears that in the early history of these works five small water-wheels were used, but, about 1838, due to the tremendous increase in the volume of business, it became necessary to secure more power with which to operate the new machines.

The foregoing were the conditions incident to the construction of the large wheel which is the subject of this paper.

Description.

The works were on the banks of the Wynants Kill, a small stream entering the Hudson River a few hundred yards below, as shown by Fig. 1. Quite close to the plant are the Burden Falls having a height of about 50 ft. and followed by a succession of rapids. The flow of this little stream was quite variable, and at some periods in the summer almost ceased entirely, being too low to furnish sufficient power. About 10 miles above this point the stream has its source in four lakes, and at the plant the total drainage area is approximately 36 sq. miles.



LAYOUT OF HENRY BURDEN & SONS' FACTORY, SHOWING RELATION OF WATER-WHEEL TO MACHINERY, RESTORED.

FIG. 1.

It is evident, therefore, that to accomplish the results sought it would be necessary to regulate the stream in some manner and also to build a wheel of sufficient size to utilize practically the total available head of about 70 ft. In attacking this problem Mr. Burden exhibited the same resourcefulness as in all his other undertakings. To accomplish the proper regulation of the stream, a lake was formed in the vicinity of the others by building a dam across a narrow part of the valley and placing therein regulating gates. It was found that this provided sufficient storage to regulate the stream and sufficient power at all times. In solving the second part of the problem, he recognized the fact that one large unit would be more efficient than five small ones, and thereupon commenced the construction of the wheel, popularly known later as "The Niagara of Water-Wheels".

It is stated that Mr. Burden commenced the construction of this wheel in 1838, and it appears in the records that, after experiencing considerable difficulty in its operation, it was rebuilt in 1851. The records show that at one time one of the journals broke off from the rosette, due to excessive heating of the bearing. This was corrected to a certain extent in the rebuilt wheel by making the journal hollow and by providing for the circulation of water over the bearings. To what extent it was rebuilt is not known, but it would seem that the same general dimensions were retained, though there was considerable alteration in the details. However, the wheel as it stands to-day is the same as the one rebuilt in 1851, save for a few changes in the penstock, which was constructed of wooden staves instead of the riveted iron pipe now standing. It is stated that, outside of the ordinary repairs incident to the operation of the wheel, the only part that was renewed was the segment gearing, which was replaced about 1882.

The dam was a few hundred yards above the falls, and the water flowed in a canal along the hillside, ending at a point about 100 ft. from the center of the wheel, as shown by Fig. 1. From this point it was carried in a brick conduit, 4 ft. in diameter, to the edge of the cliff, and there this conduit was joined to the riveted steel penstock, also shown by Fig. 1. In the canal at the head of the conduit there were head-gates.

The wheel is of the overshot type, and in general consists of a central hub to which are fastened radially 264 iron rods, 1½ in. in diameter. These are fastened to 10 by 10-in. Georgia pine timbers

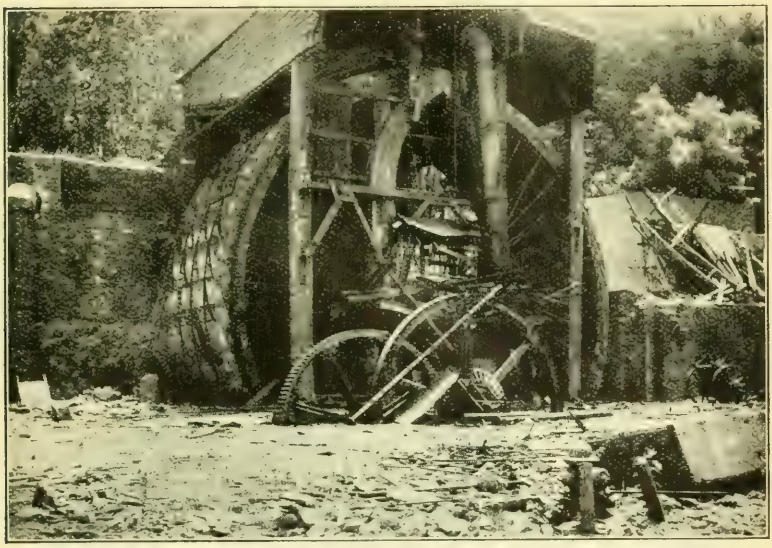


FIG. 2.—BURDEN WATER-WHEEL AFTER DISMANTLING OF WORKS, SHOWING PORTION OF ROOF.

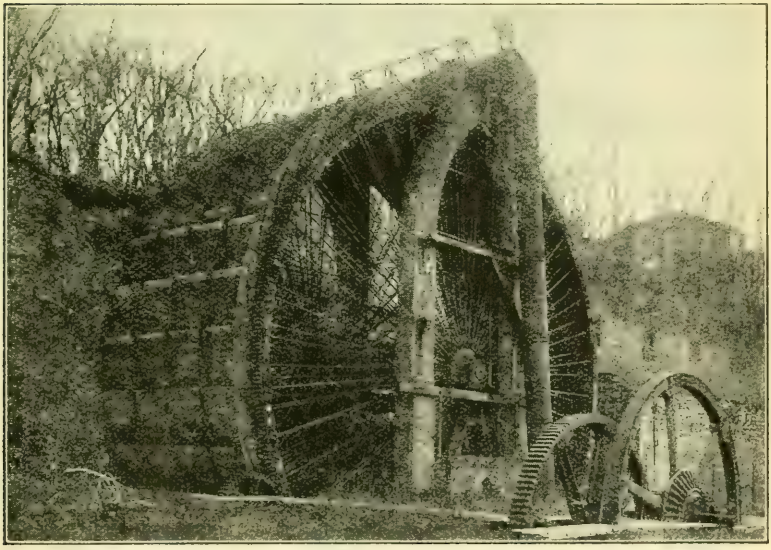
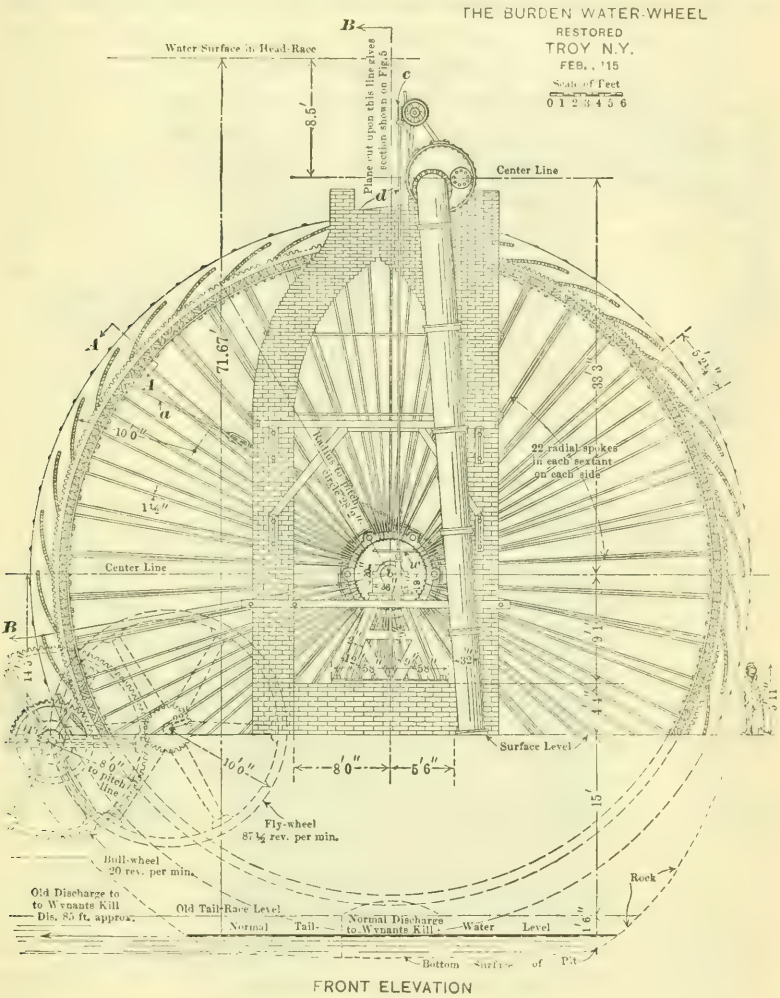


FIG. 3.—BURDEN WATER-WHEEL, SHOWING CONDITION IN 1913.

forming the soling on which the floats or buckets are built. On each side of the wheel on the soling is fastened the segment gearing which gears with two pinions keyed to the same shaft, as shown at the

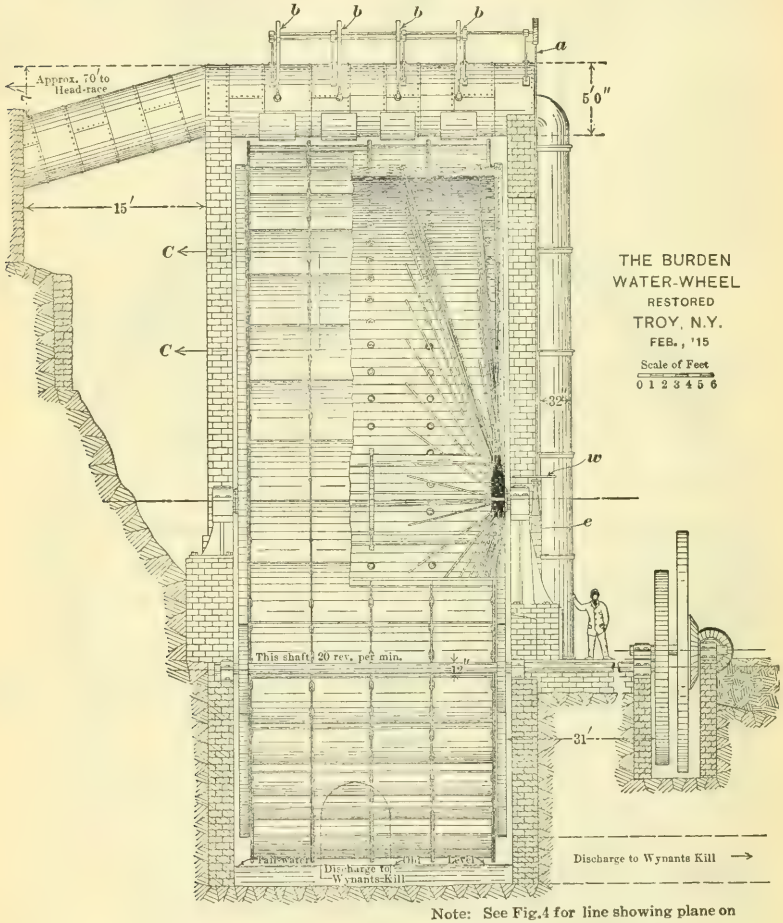


FRONT ELEVATION

FIG. 4.

ground level in Figs. 4 and 5. The wheel stands in a pit, about 20 ft. deep at the lowest point, cut out of solid Hudson River shale. The sides of the pit, along the faces of the wheel, are lined with brick to the ground level. At the center, under the bearings, the wall is

carried up about 4 ft. on the north side and 9 ft. on the south. On each of the piers there is a cast-iron base-plate on which rests a cast-iron pedestal, shown in Figs. 4, 5, and 8. The reason for the shorter pedestal on the south side is apparently due to the fact that the



Note: See Fig.4 for line showing plane on which this section is taken—Line *B-B*.

SIDE ELEVATION AND PART SECTION.

FIG. 5.

rock on that side is against the face of the cliff. Before the wheel was constructed, the cliff extended out for a considerable distance, but this was all removed in order to obtain the proper setting for the wheel.

On the pedestals rest the main bearings which support the wheel. The hub is made up of two rosettes fastened together by hollow bars, as shown in Figs. 8 and 9. Each of these rosettes is 7 ft. in diameter, and is cast integral with the journal, which is 20 in. in diameter. A section through the hub on the center line of the journal (Fig. 9) shows it to be of T-form, with the stem of the T larger where it joins the journal, and curved in outline. The top of the T is 12 in. across and radially through it are drilled three rows of holes, the outer row having eight and the other two rows seven holes to the sextant into which the hub is divided. The center holes are drilled into the body of the casting and tapped to receive the rods; the outer and inner rows are bored through the shoulders of the T and the rods are held by nuts. At each of the six points, *a* (Fig. 8), holes were drilled, parallel to the axis of the journal, and through them were placed 2 ft. 6 in. by 4½-in. wrought-iron pins, on each of which a shoulder was turned. Over each pin fits a hollow stiffening bar, 10 in. in diameter at the ends and 12 in. at the middle. These are fastened to the pins rigidly by gib and cotter, *c*. At the center of each stiffening bar is turned a shoulder, *d*, and over each is slipped the stiffening ring, *b*, and driven to a fit. Thus the whole central portion of the wheel or hub is rigidly assembled. In the actual construction of the wheel, all this was first assembled, then the first rods vertically under the hub were placed, and suspended from these was a 10 by 10-in. Georgia pine timber, 22 ft. long, which formed a part of the floor or soling on which was constructed the buckets, gears, etc. The rods, alternately on one side and then the other, were placed, and the corresponding timbers were attached. This operation was repeated until one-half the soling was in place, which formed an inverted arch. Through each of these timbers were run wrought-iron rods which served to bind the whole together and hold them in line in a radial plane. These are shown at *a* (Fig. 7). After the lower half was thus placed, a center was built for the upper half, as would be done for a full centered arch, on which the remaining portion of the soling was placed.

It will be noticed that the center row of holes in the rosettes requires that a turnbuckle be placed on each rod running to them, otherwise there would be no way to regulate their tension. The rods passing through the outer and inner rows of holes, which are bored through the shoulders of the T, permit of adjustment with nuts, as shown at

e (Fig. 9). In order to stay the wheel against any axial motion, the three rows of rods on each side are disposed as shown in Figs. 5 and 7, which is the same as in a bicycle wheel.

Figs. 6 and 7 show the method of constructing the buckets and segment gearing. The shrouding, *b*, is of cast iron, $\frac{3}{8}$ in. thick and 29 in. deep, 2 in. of it being set into the soling for the purpose of caulking the buckets. This was cast in sections and fastened together with rivets as shown at *c*. Through the shrouding passes 1-in. bolts which go through the soling and are secured by nuts and washers, as shown at *d*. There are 36 buckets, the timbers of which are of Georgia pine, 2 in. thick. Each of the timbers passes through a slot in the shrouding, and in order to caulk effectively against leakage the ends were kerfed and small wooden wedges were driven which forced the wood against the shrouding. On the inside of the shrouding and against the soling a quarter round strip was fastened, and oakum was packed behind this to prevent leakage. In after years it was found necessary to place the small angle, *e*, between the shrouding and the segment gear and to caulk between this and the shrouding, for at times the water squirted out into the shop.

In order to support the buckets at intermediate points, three cast-iron spacers, *f*, were used. These are merely frames which permit of free access from one part of the bucket to another, and yet effectively support the timbers. The same method of attachment to the soling is used as in the case of the shrouding. The spacers are cast in sections and fastened together at *g* (Fig. 7). The planks of the buckets are doweled to each other, as at *g* (Fig. 7), in order to prevent springing under pressure, which would render the oakum caulking ineffective. Another function of this doweling was to stay the buckets and shrouding from transverse motion.

The main gear is made up in segments 6 ft. long and of the dimensions shown at *h* (Figs. 6 and 7). Each of these segments is fastened to the soling by five bolts which pass clear through it and also through a $\frac{3}{4}$ by $5\frac{3}{4}$-in. bar inside the wheel. There is one of these gears on each side of the wheel, as shown by Fig. 7, and they gear with two pinions, 7 ft. in diameter and 9-in. face. These pinions are keyed to a shaft which is 12 in. in diameter and is supported on bearings on each side of the pit. The shaft extends out into the room about 31 ft. from the face of the wheel, and terminates in a large gear 16 ft.

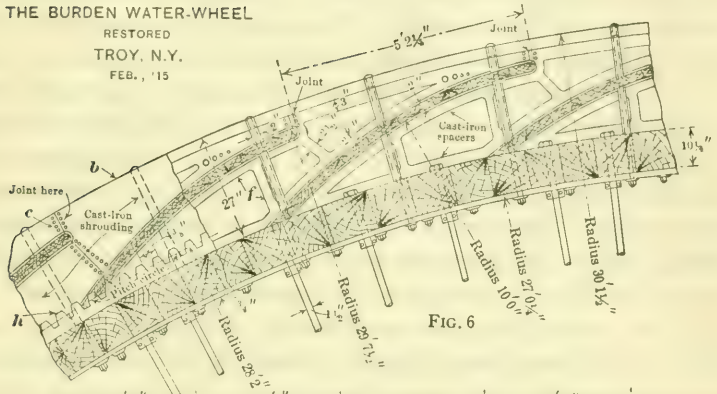


FIG. 6

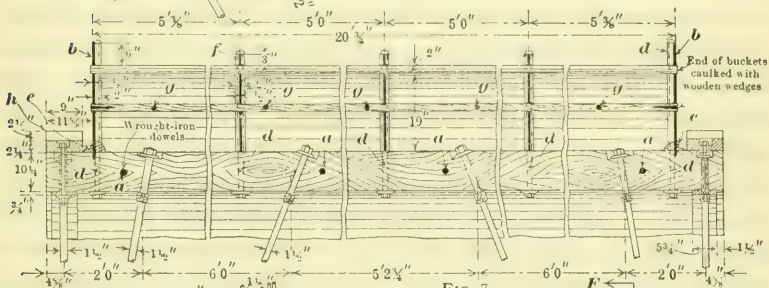
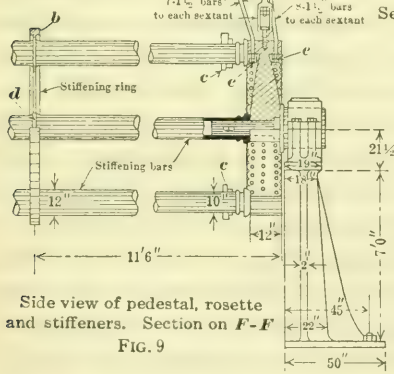
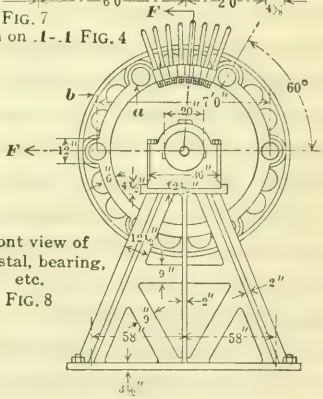


FIG. 7 Section on *A-A* FIG. 4



Side view of pedestal, rosette and stiffeners. Section on *F-F*
FIG. 9



Front view of pedestal, bearing, etc.
FIG. 8

in diameter. This in turn meshes with a 44-in. pinion which is keyed to a shaft carrying a fly-wheel 20 ft. in diameter. On the same shaft and adjacent to the fly-wheel is keyed a miter gear which meshes with another keyed to the line shaft. This line shaft ran the length of the shop, a little above ground level, and was 12 in. in diameter.

It is seen from the drawings that, the power being taken off at the periphery of the wheel, instead of at the axis, the only torque produced at the axis was due to journal friction. It is estimated that the wheel weighs about 250 tons in a soaked condition or 125 tons on each bearing. This weight was sufficient to require some means of taking care of the frictional torque produced, and accordingly the two rods, *a* (Fig. 4), were placed and provided with turnbuckles.

A very novel feature is the method of bringing the water to the buckets. The canal ends at a point about 100 ft. from the center of the wheel, and there the water is taken into a brick conduit through a head-gate. This conduit is made of three rings of brick, each ring being cemented and hooped around with wrought-iron bands. This stands to-day in apparently perfect condition. At the edge of the cliff this brick conduit is joined to a riveted iron penstock which rises and is carried over the top of the wheel, resting on brick arches close to each side. All this has fallen in, due to the failure of the foundation walls on August 22d, 1914. Up to that time the wheel was in almost as good condition as when it stopped in 1896. On the top of the iron penstock there are five iron brackets supporting a shaft which terminates in a worm wheel and has keyed to it four pinions in intermediate positions, as shown at *b* (Fig. 5). In the side of the penstock there are four rectangular orifices, equally spaced over the breadth of the wheel. Inside of each there is a sliding gate which is fastened to a stem passing up through a stuffing-box and connected to a rack. This rack gears with the pinion on the shaft referred to, and is held in gear with it by an idler which runs on a small shaft supported by the brackets. At *c* (Fig. 4) a worm engages the worm wheel and is keyed to the shaft, *d*, which runs down to the hand-wheel, *w*. This is supported by the bracket, *e* (Fig. 5) which is attached to the pedestal of the main bearing. In the operation of the wheel, a man was constantly kept at this hand-wheel and, at a signal from the shop, would admit more or less water and thereby maintain the speed constant at $2\frac{1}{2}$ rev. per min.

The discharge from the wheel was taken through a tunnel cut in the solid rock to the creek, a distance of about 200 ft. This represents the normal tail-race; but, in the early history of the wheel, a suit was brought by the owners of the property on the other side of the creek and adjoining the falls, demanding that the water be returned to the stream at the foot of the falls. Due to the numerous small rapids between this point and the point of normal discharge, this meant that the bottom of the wheel would be submerged for 2 or 3 ft., which would cut down the power very materially. This matter was adjusted subsequently, and the old discharge was opened again. The two discharge tunnels are shown on Fig. 1; the short one to the foot of the falls was closed after the adjustment of the difficulty. The vertical cast-iron pipe bolted to the end of the penstock and running down beside the pedestal was used as a penstock to a small turbine at *b* (Fig. 1). The conditions that brought about this addition are not known.

The drawings only show the restored wheel. A part of the roofing is shown on Fig. 2. The point of connection with the main factory roof is clearly shown. The wheel stood in the main room of the works (Fig. 1), and being under the same roof was afforded protection from severe weather. The rear wall of the works was built up on the edge of the cliff immediately adjacent to the wheel, and for a height of 10 ft., and the roof started from this. In this wall, and opposite the wheel, a door was placed and from this a gangway was constructed up alongside the penstock and over the top of the brick arches which supported it. The wheel, just after it was built, became quite famous, and was constantly inspected by a large number of visitors. The gangway leading to the top was opened to them by the proprietor, and quite a good idea of the magnitude of the wheel could be obtained by looking down into the shop from this height.

In severe winter weather, after a shut down, considerable trouble was caused by ice, which formed on the top and had to be chopped away before the wheel could be operated. Of course this never occurred during normal operation, as it was prevented by the heat from the furnaces. The bearings were lubricated with suet, and the caps, being hollow, were packed with this lubricant. However, the bearings became heated occasionally, and, to remedy this, water was circulated over them. The wheel was very rigid for a machine of this size, yet there was some vibration, and this eventually loosened some of the rods

and occasionally broke one. The proper tension had to be maintained at all times, in order to secure the necessary rigidity and also make the gears mesh properly. It was also the constant aim of the attendants to maintain the wheel in a uniformly soaked condition because any drying out would tend toward an unbalanced operation. This also would affect the proper meshing of the gears.

Fig. 1 gives a good idea of the general layout of the plant. In operation the following machines were driven by the wheel: one rotary concentric squeezer and muck-bar train, six 9-in. trains for rolling horseshoe and rivet iron, six horseshoe machines, fifteen rivet machines, forty-two punching machines, machine shop, roll lathes, shears, and other machines required in a rolling mill.

The wheel was in continuous operation for a period of 45 years, often running day and night for long periods; but, due to the growth of the business and the necessity for more power, the works were abandoned in 1896 and the machinery was moved to the steam mill which is a short distance away on the banks of the Hudson.

The wheel is rapidly falling into decay, and before long its destruction will be complete. The supporting piers have failed, and the wheel has settled into the pit. The brick arches also have fallen, and the iron penstock is resting on the top of the wheel, broken away from its connection with the brick conduit and vertical cast-iron pipe. As one stands and looks on this, he is forcibly reminded of the irresistible advancement in engineering lines, there is such a contrast between this huge motor and the compact, high-power units of to-day.

Power and Efficiency.

Thus far nothing has been said of the power and efficiency of this wheel, but, as this is of interest, the writer has worked up, from known conditions of operation, the curves shown. A brief explanation of the method of deriving these curves is given.

To determine the power and efficiency of an overshot wheel, for different conditions, by the methods given by Weisbach, would be very troublesome, and so the writer has resorted to a graphical solution. It will be assumed at the outset that each bucket is completely filled at the top of the wheel, and then the intermediate conditions can be readily determined.

It is clear that if the bucket is initially completely filled, it will start spilling at once and continue to do so until it is completely discharged. To determine the power developed under these conditions, an integral expression of the form,

$$\text{Horse power} = \int \frac{W\ dH}{33\ 000},$$

must be evaluated. In this expression there are two variables, and, from a well-known principle of the calculus, if for all values of H we can determine the corresponding values of W, it can be evaluated. It is evident that, for every position of the bucket, the water contained therein is a definite quantity which fulfills the foregoing conditions. By making a drawing of one of the buckets to a large scale, and considering it in ten consecutive positions from the top around to the lowest point, would mean that by drawing lines tangent to the lip of the bucket every 10°, the area within each and the outline of the bucket is a measure of the quantity of water contained. The center of gravity of each section thus determined, was plotted, and the head above tail-water measured. The integral was changed to the form,

$$\text{Horse power} = \int \frac{C\ A}{33\ 000}\ dH,$$

in which C is a constant and A is the cross-sectional area of the water in the bucket. In Fig. 11, the term $\frac{C\ A}{33\ 000}$ was plotted as abscissas against H as ordinates.

This takes care of the water from the topmost bucket to the tail-race level and in order to include the effect due to the impulse of the entering jets from the four orifices the following expression was used:

$$\text{Effective head on top bucket} = (V \cos.\ O - u)\frac{u}{g},$$

in which V is the absolute velocity of the entering water, O is the angle the entering water makes with the tangent to the bucket, and u is the peripheral velocity of the center of gravity of the water in the bucket. V was determined by the Bernoulli equation, and found sensibly constant for different rates of discharge, because the area of the orifices is small compared with that of the conduit. By consulting the curve, Fig. 10, it will be seen that by dividing the greatest abscissa

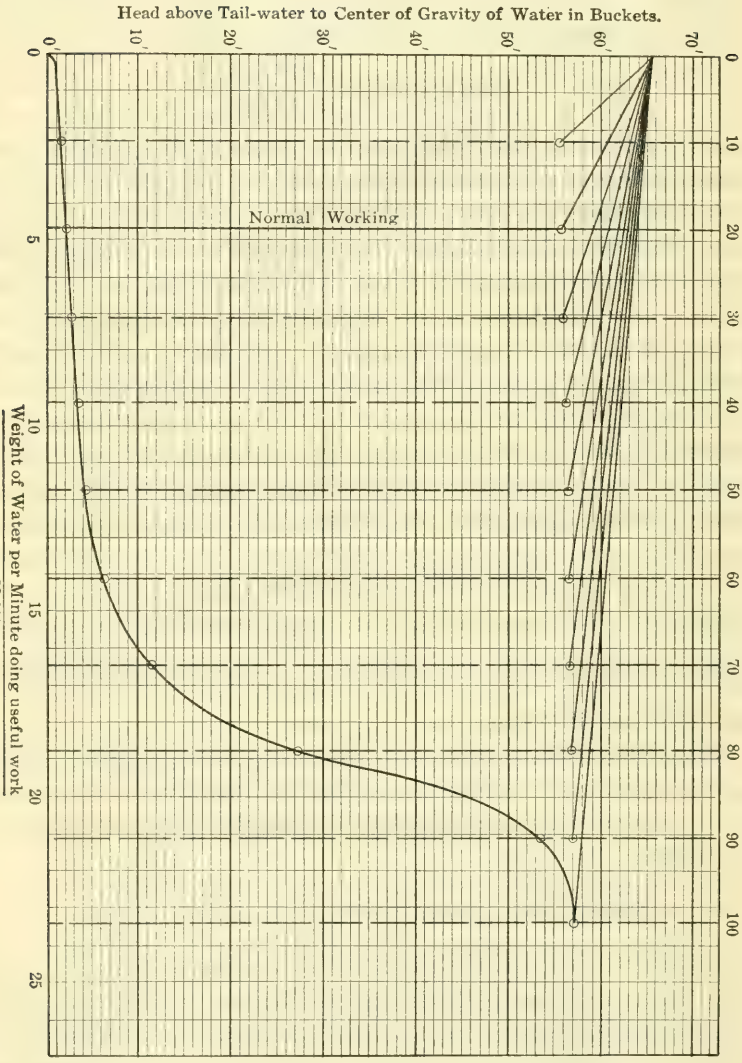

Head above Tail-water to Center of Gravity of Water in Buckets.
0′ 10′ 20′ 30′ 40′ 50′ 60′ 70′
0 5 10 15 20 25
Normal Working
Maximum Capacity of Bucket, Expressed as a Percentage.
0 10 20 30 40 50 60 70 80 90 100
Weight of Water per Minute doing useful work / 33 000

FIG. 10.

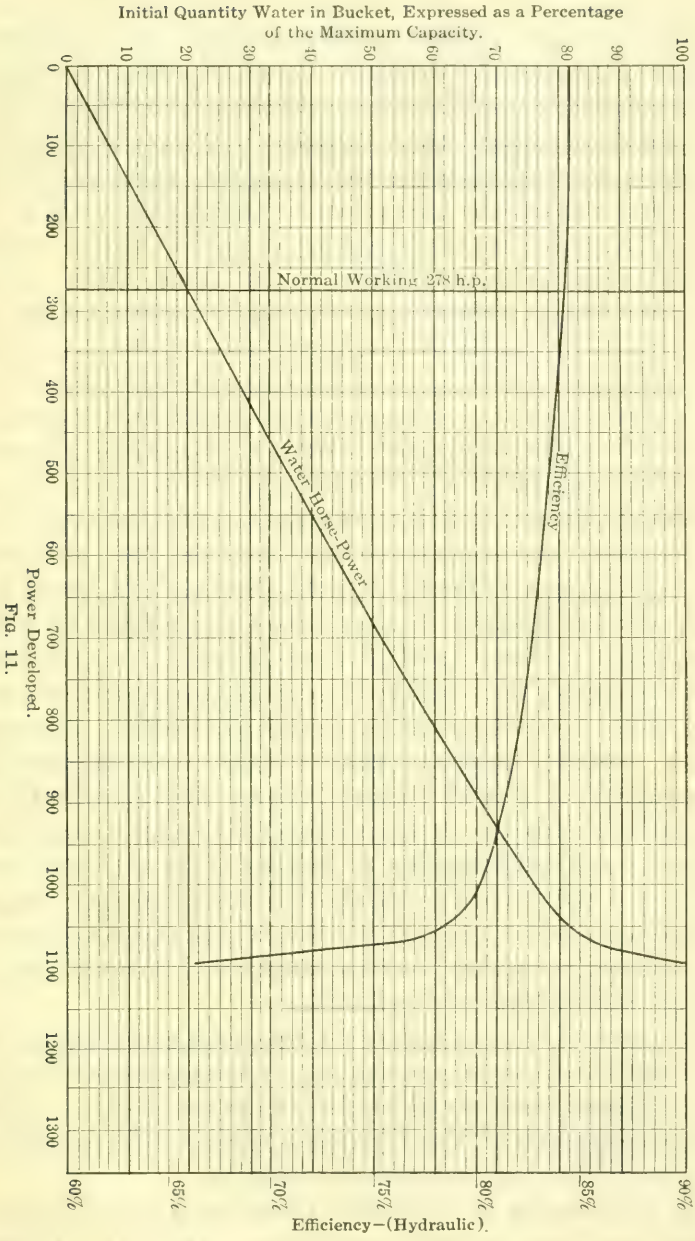

Initial Quantity Water in Bucket, Expressed as a Percentage of the Maximum Capacity.
0
10
20
30
40
50
60
70
80
90
100
Normal Working 278 h.p.
Water Horse-Power
Efficiency
Power Developed.
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
60%
65%
70%
75%
80%
85%
90%
Efficiency—(Hydraulic).

FIG. 11.

into ten equal parts by vertical lines, where these intersect the curve will give the height at which spilling commences for that quantity of water in the buckets. The circles on each of these lines represent the height of the center of gravity of the water in the highest bucket with this percentage of filling. Therefore, to the left of any one of these lines, the area included between it and the vertical axis and the lower curve will give the horse-power developed by the action of the water on the wheel, and that to the left of this line and the vertical axis and below the curve will give the lost energy due to spilling out of the buckets. Now, in order to take into consideration the energy due to the impulse at the top it will be observed that if each of these vertical lines be carried up a distance above the circles equal to the effective head on the bucket due to impulse then the area to the left of this extra height and the vertical axis will give this power. In order to simplify the work a little, instead of doing this, a point equivalent to twice the effective head was found on the vertical axis and lines were drawn connecting the circles and this point which gives the same area as before. It will be observed that we are now in a position to integrate for power under all conditions of initial filling, and at a constant speed of 2½ rev. per min. From this curve, the curve, Fig. 11, was constructed, and lines showing the conditions under which the wheel normally operated were drawn. This gives 278 h.p. at 84.25% efficiency. This, of course, means the hydraulic efficiency only, as a great quantity of power must necessarily have been lost in the gearing. It must be borne in mind that the possible maximum power was determined by the head on the orifices and their greatest area. This would be a little above the normal power because the gates were almost opened to their limit. The curves are instructive, however, in showing what the wheel could do with sufficient water, as well as showing its high efficiency.

The matter of centrifugal action was investigated, and it was found that the center of curvature of the surfaces of the water in the buckets was located on the vertical axis of the wheel, at a height of approximately 475 ft. above the center of the wheel. Its effect on the power of the wheel, therefore, was negligible.

The writer wishes to acknowledge the following sources of information, from which he had drawn freely: "Troy's One Hundred

Years", "Henry Burden", by Margaret Burden Proudfit; and graduating theses at the Rensselaer Polytechnic Institute by Frederick Grinnell, '55, George F. Kirby, '57, Charles McMillan, '60, Horace Crosby, '62, and Abraham B. Cox, '67. Much valuable material was obtained from these theses, as they were written while the wheel was yet new. The writer is also indebted to Messrs. J. A. Leskie and R. S. McEwan, both of whom were workmen at the old plant.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA

Discussion.*

By Messrs. E. L. Sayers and A. C. Polk.

Messrs. Sayers and Polk.

E. L. Sayers† and A. C. Polk,‡ Members, Am. Soc. C. E. (by letter).—The writers are pleased at the extent of the individual discussions on this paper, and will be glad to answer to the best of their ability the many questions brought up. Their only regret is that there was not a full discussion on the more general topic of modern methods of power development.

Shortly after the paper was published, one of the writers received a letter from Clemens Herschel, M. Am. Soc. C. E., asking a number of questions which he suggested could be answered in the closing discussion.

Mr. Herschel wished to know:

1st.—What reduction of head would there be in times of flood, and why were not fall increasers used?

2d.—What was the method of determining the size of the installation?

3d.—He thought that too much importance has hitherto been assigned to the "nature and extent of the power market" and to its consequent load factor. He thought that load factors are transient things, with a consequent tendency to disappear, or to become equal to one, in course of time, and the progress that time brings with it;

* Authors' closure of the paper by E. L. Sayers and A. C. Polk, Members, Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Macdonald, W. Va.

‡ Cohoes, N. Y.

Messrs. Sayers and Polk.

but that power-plants, and hydraulic works generally, are built to last as long as any known handiwork of man.

Taking up Mr. Herschel's first question: The maximum back-water effect which may be expected annually is 6 ft., reducing the maximum head to 62 ft., at which head the output of the wheels will be 16 000 h.p. As noted in the paper, there will be below Lock 12 another dam at the site of Lock 14, on which there will undoubtedly be flood-gates for controlling the level of the lake formed by it. The maximum available head now is 72 ft., which will be reduced to 68 ft. by the construction of this lower dam.

Under the greatest flood condition known to have existed, the head would be reduced to 54 ft., at which head the output of each wheel would be reduced to 13 500 h.p.

The matter of back-water was studied carefully from every angle, and the figures were checked by the establishment of gauging stations on the river and by the testimony of the oldest inhabitants.

Fig. 58 shows the rating curve constructed as the result of a great many readings, made first from boats and later from an overhead cableway stretched across the river at a point about ½ mile below the dam. The river at this point is considerably narrower than at the site of the dam, and on this account the fluctuations of level there are more pronounced than in the tail-race. At first, the readings were made with the company's meter alone, but, later, they were taken with both this meter and one belonging to the U. S. Geological Survey which, immediately prior thereto, had been calibrated at Washington.

With reference to fall increasers, it should be noted that each of these units uses 2 600 sec-ft. of water under full head, and fall increasers, though no doubt practical on relatively small installations, would not be practical in the case of such large units. Another vital objection to their use is that they involve increased investment and the establishment of large gates under high heads, with the consequent leakage, while the limiting factor in most hydro-electric developments is the careful conservation of the low-water flow. Another consideration involved is the fact that for periods of low water it was necessary to provide a steam auxiliary plant as a standby. This being the case, the construction of fall increasers would be an economical error.

With regard to the second question, relating to the size, it may be stated that the installation finally chosen was selected because one which at 80% power factor would produce 45 000 kw., under a safe assumption as a load factor, was calculated to be profitable. The wheels will carry this load under a head of 56 ft., and when the head is less it will be necessary to use the steam plant.

Using the conditions of 1897 for purposes of computation, a regulated flow of 2 650 sec-ft. can be obtained, using a draw-down of

Messrs. Sayers and Polk.

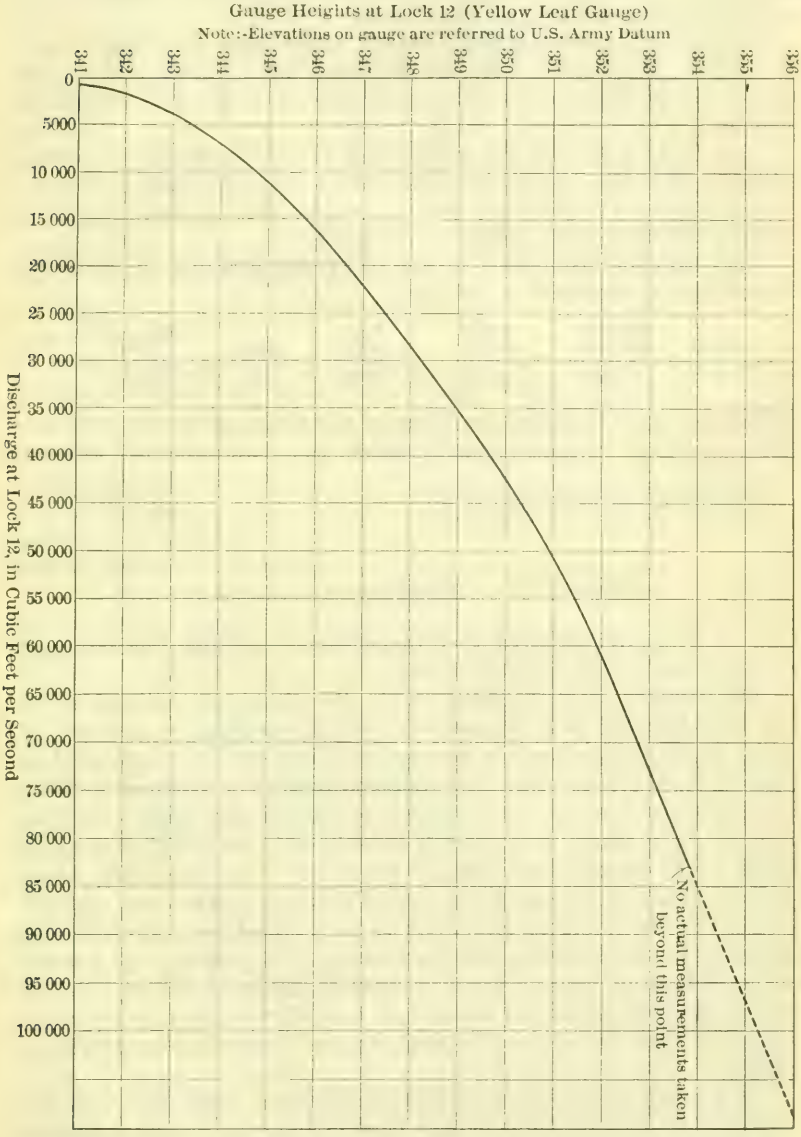

FIG. 58.

Messrs. Sayers and Polk.

10 ft. on the pond level. At the present head, this equals 286 000 kw-hr. per 24 hours. The Gadsden steam plant, at 94% power factor, will produce 225 000 kw-hr. per 24 hours.

The primary power available for sale under these conditions, therefore, would be 511 000 kw-hr. per 24 hours, and, at 50% load factor, this would be equivalent to an installation of 43 000 kw., maximum demand.

As a matter of fact, the actual load factor is greater than 60%, although 50% was assumed, which means that the plant will have a certain amount of secondary power to sell, or that the steam auxiliary must be extended when it becomes desirable.

The ultimate installation of six units will produce, in 24 hours, 933 000 kw-hr., estimated as follows:

Units.		Kilowatt-amperes.		Power factor.		Load factor.		Hours.		Kilowatt-hour.
6	×	13 500	×	80%	×	60%	×	24	=	933 000

This is available from the river 66% of the time, and it will require a steam auxiliary of approximately 30 000 kw., determined as follows:

Total output in 24 hours	933 000 kw-hr.
Lock 12 production, dry period	286 000 " "
	647 000 kw-hr.

$$647\,000 \div 24 = 27\,000 \text{ kw., say, } 30\,000 \text{ kw.}$$

It is to be remembered that the locality in which this power is marketed has a very low-priced coal.

The writers do not agree with the apparent contention of Mr. Herschel that load factors are to be disregarded, or assumed as equal to unity. In certain communities it would be impossible to attain this condition. In a New England district of factories working 8 or 10 hours per day, and where the lighting and night street-car loads would be small, it is entirely reasonable to assume that the load factor would remain small for all time; but in a community where electric furnaces, cement mills, electro-chemical works, etc., abound, the load factor would be large, and after the load was once taken by the plant it would remain about stationary. Of course, it is possible to conceive of a locality where a large diversity of industry would permit the commercial department of a power company to select its customers with the view of raising the load factor to approximately unity.

In general, it may be said that the designer must deal with the market as he finds it, not as he may hope to make it. In one particular case, however, though the market to which the Lock 12 power is brought is a new one for electric power, and for that reason is not well defined,

it has certain limits of character, and it can safely be said that the load factor on the Lock 12 plant will never approach closely to unity. Messrs. Sayers and Polk.

Mr. Harlow criticizes the writers on the score that they have not written the paper so as to answer the question "Will it pay?" It is unfortunate that this question cannot be answered in such a way that it can be supported with figures. However, as this was the fundamental study on which the whole project was based, and the plant is still in operation, it would seem that the question is automatically answered. The company has weathered two or three of the worst financial years in recent history, without failing to meet its obligations; at the end of one year of operation it has already under contract more than enough business to pay its fixed charges, and has in immediate prospect enough business to pay fair returns on the money invested.

After noting the fact that the plant has been in operation a year, Mr. Harlow again asks "Has it paid?" and "What was the cost * * * of the hydro-electric plant per kilowatt on the station switch-board?" and also "What was the cost of the steam auxiliary plant per kilowatt on the station switch-board?" He notes that the cost per kilowatt should be based on the actual annual output of each plant. This method of calculating the cost might be fair if the plant had a full load waiting for it on the first day it was put into operation, but this was not the case; and it very seldom is. The writers cannot see, however, how such figures of cost could be more than matters of idle interest. If compared with operating cost and revenue they would answer Mr. Harlow's first question, but what then? They would be of no use to designers and operators of other plants, for, at what value to the designers of the Lock 12 plant would be the cost per unit of output of the Hales Bar or the McCalls Ferry plants or even of the Keokuk plant; or of what value would the same figure for Lock 12 be to the designers of another plant, say below McCalls on the Susquehanna or at Muscle Shoals on the Tennessee? The writers agree with Mr. Harlow, to a certain extent, that detailed costs would be valuable to designers of other plants, but the difficulty is that so many factors enter into costs that, unless one is acquainted with all the details of the construction, the figures are practically valueless. If one had in detail the costs of the material entering into the concrete, and the reasons for the cost being such, and the cost of the labor of mixing, handling, and placing the concrete, with details as to rates of wages, length of working day, number of shifts worked per day, layout of plant, flood conditions, etc., etc., the figures might be studied and prove of some value. Even the cost per rated horse-power of the turbines, or the cost per kilowatt of the generators, are of no value for comparative purposes, for the reason that there is at times keen competition in

Messrs. Sayers and Polk.

seeking contracts for the supply of such equipment, and, besides this, the personality of the representatives of the contracting parties enters into the case in no small degree.

Mr. Harlow's question with regard to the selected size of installation has already been answered. He asks:

"At the plant in question, can more than one unit be operated at the extreme low-water flow, and if two must be run to get the use of all the water flowing, is not the efficiency being reduced much below that expected?"

In reply it may be stated that the most economical plan of operation during the low-water period would naturally be to operate the steam auxiliary plant to take the base load and to use the hydro-electric plant, taking advantage of the permissible 10-ft. draw-down, to carry the peak loads. This being the case, the loss of efficiency which Mr. Harlow fears would be reduced to a minimum, and would be of short duration daily, because two, or even three, units might be required to carry these peaks. A small amount of adjustment between the auxiliary and hydro-electric plants at these times will permit of using the water to the greatest advantage.

With reference to the actual efficiency of the turbines, the engineers who designed and built the plant made provision for a test of the actual efficiency, having placed in the concrete foundations the necessary ducts for the test of the apparatus, and having constructed piers at the limits of the tail-race for the test weir. Unfortunately, at the time of completion, high water prevailed in the river, and it was necessary to leave the test for the present operating management.

The writers are advised by the present management that the test of efficiency will be made by the "titration" method. This consists in introducing into the head-water a salt solution of known density at a known constant rate. The tail-water is sampled and the density of the salt solution again determined. The ratio of the density of the initial salt solution to the tail-water solution multiplied by the rate of flow of the initial salt solution gives the discharge of the water through the wheels. This, of course, presupposes a thorough mixture of the salt solution with the water flowing through the wheels. The details are matters of careful mechanical and chemical adjustment. Remarkable accuracy is claimed for the method.

The art of turbine building has come to such a point in recent years that it is not quite as unsafe as one might infer from Mr. Harlow's remarks to "depend on the Holyoke tests of a (comparatively) small turbine and then the guess of the builder that the full-sized turbines will 'fill the bill'." The guess of the builders has been so successful in the design of some recent turbines that the guaranteed efficiency has not only been exceeded, but the actual efficiency has

exceeded by wide margins the Holyoke tests of the model runner. The Appalachian Power Company's units, which are of the same model as the Lock 12 units, attained an actual efficiency in test 4% greater than the maximum efficiency secured in the Holyoke test. This is a matter of unprejudiced record. Messrs. Sayers and Polk.

"The hydro-generator is now going toward the vertical; shall we, after a while, swing back to the horizontal?" Probably not. The tendency is toward large units to save cost per horse-power installed, and large units and horizontal shafts do not go well together. First, because it is possible to obtain higher efficiencies in large units with vertical shafts; second, because in most cases it is possible to design a much smaller power-house; and third, because the design of a satisfactory horizontal shaft for such large units as those at Lock 12 is practically impossible, due to the very heavy flexural stresses.

Mr. Coombs complains in his discussion as follows: "Unfortunately, the authors have followed the usual practice and dismissed the subject of line construction in a very sketchy way." The complaint is probably justified in a way, but the avowed purpose of the paper was to describe the design and construction of the Lock 12 dam and power plant. The same complaint might be urged against the manner in which the construction of the Gadsden steam plant, the sub-stations, and the study work on proposed developments were treated in the paper. There were very interesting features in all this work, but a treatment of them all in detail would have lengthened the paper beyond all reason. Mr. Mitchell, the present Operating Manager of the company, who was the Electrical Engineer in the construction organization, has answered the points brought up by Mr. Coombs.

Replying to Mr. Cooper, the thrust-bearings, as located at the top of the generator frame, seem to the writers to be satisfactory, and thus far no trouble has been traced to this arrangement. In fact, one of the writers has just completed the installation of three of these bearings on 10 000-h.p. machines, and no trouble is expected from them. There was some trouble at Lock 12, due (as stated to the writers) to foreign matter getting into the oiling system. The writers agree with Mr. Cooper in his appreciation of the Kingsbury type of bearings, and only disagree with him in the extent of their praise. Mr. Cooper believes that these are the coming bearings for vertical shafts under heavy loads, but the writers believe that they were coming some 4 years ago, and that they have now arrived. Answering Mr. Cooper's question, no machine work of any kind was done on the castings in the field, and none was necessary. There was no trouble whatever from the castings being thrown out of line by the concrete. A speed ring was first set exactly and grouted in firmly, then all other castings, which had been previously

Messrs. Sayers and Polk.

fitted to the speed ring at the factory, were placed and the whole was grouted, the concreting operations taking 2 days.

Mr. Homberger says "With first-class design, vertical-shaft turbines of this particular type, of such huge dimensions, could hardly be otherwise [than self-contained]; each, of necessity, must be a homogeneous structure, or it would be impossible to make a satisfactory job of the installation in the plant." This statement is decidedly in error. The Keokuk units, which structurally are larger than the Lock 12 units, were built with incomplete pit liners. At Keokuk these were made up of rolled steel, and they required an immense amount of expensive fitting in the field. In fact, all units (or practically all) that were built prior to the construction of those at Lock 12 were constructed with incomplete pit liners. Mr. Taylor deals fully with this feature of the design, and it is hardly necessary to add to his remarks.

The taper fit of the runner on the shaft, referred to by Mr. Homberger, was selected primarily because it permitted the easy removal of the runner by the use of screws. This facility of removal may be worth considerable money in an emergency, and in reality costs very little more than a well-executed parallel press fit.

Mr. Homberger's questions with regard to the selection of the runner for the turbine and the location of the point of maximum efficiency are well answered in Mr. Taylor's discussion and by the curves of efficiency which he has shown.

The reason for the selection of vertical rather than horizontal shafts has been dealt with in the paper, and has been again referred to herein. It would be impossible, without devoting great space to the subject, to give the detailed studies which were made prior to the selection of the vertical-shaft units.

The writers fail to see the "prime advantage" of the horizontal-shaft unit in so far as greater accessibility is concerned. It is a question as to whether or not a scroll casing built of a number of parts, always remembering that these are units of enormous capacity, can be dismantled more readily than a vertical unit, with the assistance of a 100-ton crane. Furthermore, if one is to believe Mr. Homberger's statement in the paragraph criticizing the taper fit of the runner on the shaft, where he implies that the runner will seldom be removed, this supposed "prime advantage" is not of much importance.

Mr. Taylor has contributed a very thorough discussion on the modern design of turbines and on various correlated questions, and the writers feel sincerely indebted to him for thus supplementing the paper. His criticism of the fact that no independent hand-control of governor was provided is slightly in error; it is not necessary to shut down a wheel to repair the governor. The mechanism is arranged so that the dash-pot can be cut out and hand operation used. The pilot-valve, it is true, cannot be cut out, but there is little about it

to get out of order, and even then the addition of two ordinary stop-valves will allow the removal of the pilot-valve. This addition, however, seems to be of questionable advantage because the careless closing of one of these valves would permit a runaway. When running on hand-control, with these cut-outs made as outlined above, the gates are set at a constant opening, and a second turbine running on the governor control will take the variation in load. After nearly a year's operation it has not been found necessary to take out the pilot-valve on any unit for any purpose whatever. Messrs. Sayers and Polk.

Replying to Mr. Miller's remarks, the writers offer the following: There is no doubt that great improvements have been made in the type of buttons and fall-rope carriers used at Lock 12, there being considerable room for improvement in this design and the writers can well understand that the type of carrier and button used at Lock 12 could not be used at a speed of 1 850 ft. per min. The new carriers built to replace the old ones followed the general design of the old ones, but, as stated in the paper, they were of a lighter material, and were proportioned a little differently. Unfortunately, no detailed record was kept of the changes made, the carrier finally used being the result of several small changes in the general make up. Since the completion of the Lock 12 work, the writers have seen carriers built on the shock-absorbing principle, and consider them a great improvement. In the 18 months of service at Lock 12 each cableway wore out two button lines, so that the improvement along this line is indicated by Mr. Miller when he states that button lines now frequently last 4 or 5 years, this being a longer period than has ever come under the writers' observation. The maximum run of the cableways at Lock 12 was about 450 cu. yd. of concrete per cableway, per 10-hour day, the average travel being about 600 ft. The writers cannot agree with Mr. Miller that no other device will handle excavating, large rock, and concrete as well as a cableway. Mr. Miller has probably never stood under a long cableway and attempted to place a large stone swinging up and down on the end of the fall line, or attempted to land a skip loaded with excavation on a dumping platform for loading into a car where the materials must be disposed to a dump. It is an exceedingly slow method, much time being lost in spotting, and the position for the man on the platform unhooking is precarious. The writers do not agree that it is the best method for handling concrete, either, their experience being that a dinky and cars to a derrick are about 25% faster than cableway to derrick. For other general work around dam construction, it is certainly a most handy tool, and one that should always be given its place in the plant.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SUBMERGED PIPE WORK AT PORTLAND, OREGON

Discussion.*

By Messrs. W. R. Phillips and D. D. Clarke.†

Mr. Phillips.

W. R. Phillips,‡ M. Am. Soc. C. E. (by letter).—The selection of the design of the ball joint for the 30-in. submerged pipe line across the Willamette River, described by Mr. Clarke, was governed largely by the great importance of having the work of assembling completed in the shop, for there only could the workmanship be thoroughly inspected.

This involved a design which would be least likely to be injured in handling between the shop and the work, as well as in connection with the work—a design which would have a uniform distribution of lead of minimum thickness; which would have the greatest possible surface of caulked lead in contact with the finished cast-iron surface, a portion of which would be at the zone nearest the great circle; which could not be forced apart; which, in case of necessity, would permit of disassembling with comparatively little trouble; and which would permit of a considerable movement of the ball without uncovering the lead. The joint adopted seemed to meet these requirements better than, for instance, the Ward type.

The rig for finishing the balls—made in the shop which had the contract for furnishing the joints—was attached to the ways of an ordinary lathe. It consisted of a swinging arm which carried a tool rest pivoted under the point where the axis of the ball intersected the plane of the great circle. A tool set on a level with the lathe centers would swing horizontally in an arc of the same radius as that of the great circle, and thus insure a spherical turning of the ball. The

* Discussion of the paper by D. D. Clarke, M. Am. Soc. C. E., continued from April, 1915, *Proceedings*.

† Author's closure.

‡ Portland, Ore.

Mr. Phillips.

tool was fed to its work by using a link connecting the swinging arm with the lathe carriage, so that the feed could be handled in the same manner as in straight lathe work. The speeds used, and the cuts, were the same as in any work of similar size and material. Great care was taken that the finishing cut should leave the work true and smooth, and the lathe tool alone was used to finish the surface of the ball. Tests showed that the balls were practically perfect spheres.

In making up the joint, the socket was set level, with its flange resting on a blind flange. The ball was set into the socket so as to rest on the ridge shown on Fig. 1. The lead was poured, and a light sledge was used in caulking. A $\frac{3}{8}$-in. tube of heavy rubber was used as packing between the flanges. The ring was leaded and caulked in the same manner as the other piece. When the work was completed, a blind flange was placed on the upper flange of the ball, and the joint was subjected to a test pressure of 225 lb. per sq. in., which pressure it had to withstand without sign of leakage. It was noted that as the pressure was released, the ball settled perceptibly into the socket, thus assuring the freedom necessary for rotation without damage to the lead.

After the joint was tested, nothing more was done to it, as it was then considered to be finished. It is probable that the pipe extended under pressure, and took up the play which developed when the joint was tested.

In lowering the pipe into its bed, care was taken that the sections—each 60 ft. long between the joints—were not deflected with relation to one another to such an extent that the lead would pass off from the finished surface of the ball.

Mr. Clarke.

D. D. Clarke,* M. Am. Soc. C. E. (by letter).—The writer desires to express his thanks for the interest taken in the discussion of his paper by the several members of the Society, who have commented on the work done in Portland, or have offered suggestions as to improved methods based on experience in other localities.

In reply to the suggestions offered by Mr. Wiggin, the writer takes pleasure in supplying additional details regarding the methods adopted for the Portland work, and measurements, costs, etc. The omission of these details from the paper was due mainly to the desire not to encumber it with matters which were thought to have only a local application, and hence would not be useful in other places. For the benefit, therefore, of those who wish additional information on the several points which have been raised, the following data are submitted.

Mr. Wiggin states:

"Prices, for example, are given sometimes in lump figures. A price per pound, or details permitting of easy translation to price per pound, would be of much more general use."

* Portland, Ore.

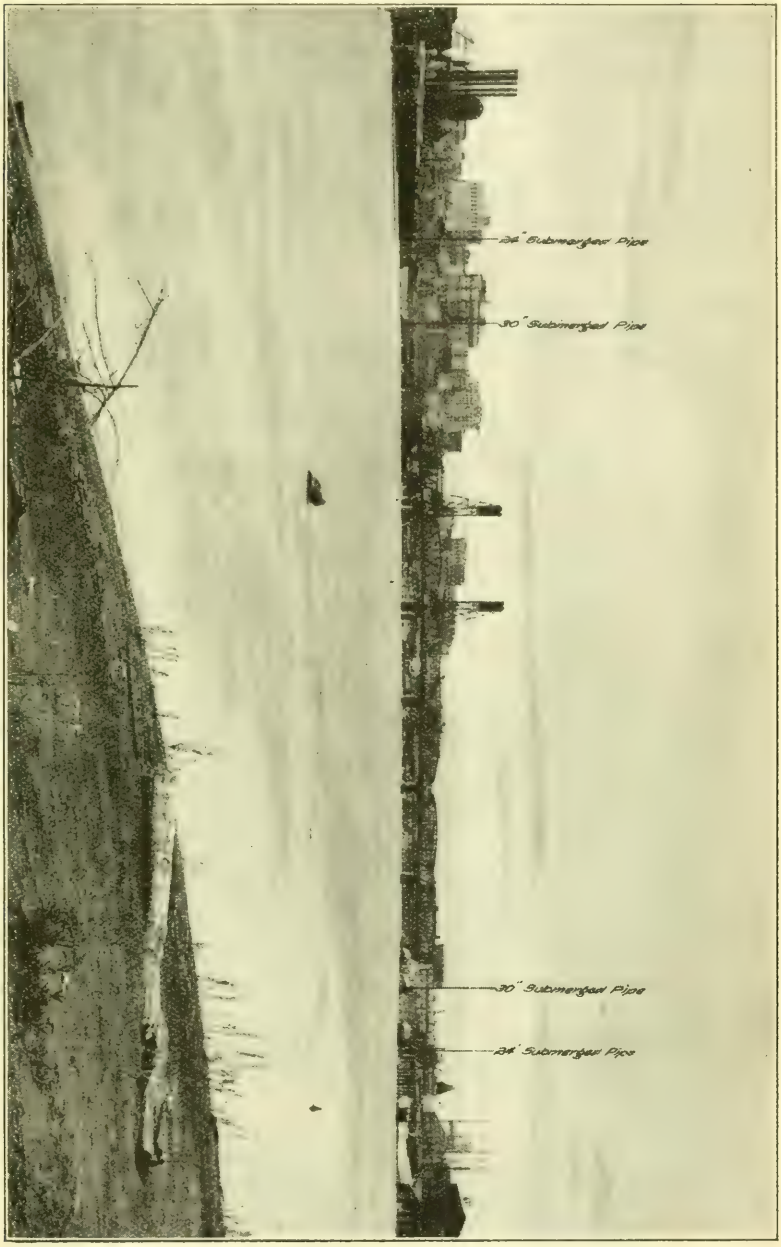


FIG. 8. UPPER PORTLAND HARBOR, LOOKING NORTH FROM LOWER END OF ROSS ISLAND. HAWTHORNE BRIDGE IN CENTER.

Mr. Clarke.

As a rule, the prices given for the ironwork were those quoted for the several units named in the contract. In the case of the 28-in., cast-iron pipes laid in 1894,* the payment was on the basis of $31.4275 per lin. ft., the weight or price per pound not being given.

In an article† by the Ohio Pipe Company, of Columbus, Ohio, the contractors for the manufacture of the 28-in. pipes, describing the process of manufacture, the statement is made that the estimated weight of each pipe section, with sleeves bolted on, was 10 000 lb. The actual weight of these joints, as found when the line was removed, approximated 9 628 lb.

Assuming the latter weight and a laying length of 15 ft. 8 in., gives a price of $102.28 per ton of 2 000 lb. for pipes and sleeves; and the contract price for pipe-laying, $2.50 per lin. ft., is equivalent to $8.14 per ton of 2 000 lb.

For the 24-in. line, the sixty flexible joints were contracted for at $260 each. The calculated weight of these joints, including bolts and lead, is 5 365 lb.; the price per pound, therefore, approximates 4.84 cents.

The pipe used for this line was $\frac{3}{8}$-in., lap-welded steel with cast-iron flanges at each end of the sections, approximating 20 ft. in length.

Estimated weight of pipe per linear foot	94.62 lb.
Estimated weight of two flanges per foot of pipe	53.00 "
Total weight	147.62 lb.
Contract price per foot of pipe and flanges	$8.50
Estimated cost per ton, therefore	$115.18

For the 30-in. line (submerged portion), thirty flexible joints were purchased for $345 each. The weight of each joint, ready for laying, is 5 720 lb., the estimated price per pound being 0.0603 cents.

The 30-in., lap-welded steel pipes, galvanized and asphalted, with cast-iron flanges, cost $14 per lin. ft. These pipes were made of $\frac{7}{16}$, $\frac{1}{2}$, and $\frac{9}{16}$-in. plates, the average length of each pipe being about 16 ft. Of this pipe 2 081 lin. ft. were purchased.

Total weight of pipe, including flanges	464 469 lb.
Weight of 3 500 bolts with hexagonal nuts	17 730 "
Total weight	482 199 lb.
Average weight per foot	231.7 "
Average cost per pound	$0.0604

* *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 257 (1895).
† *Iron Age*, January 10th, 1895.

Mr. Clarke.

The 30-in., cast-iron pipes used for shore connections cost $11.10 in place in the trench. The estimated weight, per 12-ft. section, is 4 700 lb.* The estimated price per ton of 2 000 lb. is $56.63 for pipe in place, including lead and oakum.

Mr. Wiggin asks how the excavated material was measured.

When the 28 and 24-in. lines were laid originally, the trench was excavated with a ladder-dredge, the excavated material being deposited on barges alongside the dredge. The barges were towed away later, and the dredged material was used for filling behind the dock line in the lower portion of the harbor, as already explained. The contractors were paid by the cubic yard of material actually dredged and carried away, as measured on board the barges transporting it. For the most part, the trench was excavated in fine gravel and sand, which took a flatter slope under water than was anticipated, and, in consequence, the dredging quantities exceeded the original estimate. This risk, however, was assumed by the City.

The dredging for the 30-in. line was done partly by suction-dredges and partly by a dipper-dredge, on a *per diem* basis. The material removed by the suction-dredge was deposited behind the line of an adjacent dock, and the quantities dredged were estimated from soundings taken along the line of the completed trench. The total cost of this part of the work was $16 160.14, and the estimated yardage removed was 102 655, making the cost $0.1574 per cu. yd.

The material removed by the dipper-dredge was deposited on a barge moored alongside, the barge being unloaded by the dredge owners at their own expense, and the material was used for making street embankments in an adjacent district.

The total cost of this part of the work to the City was $4 158, and the scow measurement of yardage was 29 700 cu. yd., making the cost to the City 14 cents per cu. yd.

It has already been stated that the quantities of trench excavation exceeded the original estimate. This estimate (57 000 cu. yd.) was based on a bottom width of 6 ft., with side slopes of 1:2, and an average depth of 20 ft. When the work was done, however, it was found that the material in places would take an under-water slope of approximately 1:5 or 1:6.

Although it is true that the dredging quantities largely exceeded the original estimate, the total cost of the work did not, the estimated price per yard being taken at the contract rate for the 28 and 24-in. lines previously laid, namely 40 cents per cu. yd. for dredged material and its disposal (the regulations of the U. S. Engineers requiring the trench material to be removed entirely from the river channel). Instead of 40 cents per cu. yd., however, the work actually cost only a little

*Class "F", New England Water Works Assoc. specifications.

more than one-third of that, on account of the superior dredging facilities, etc. Mr. Clarke.

Mr. Wiggin brings up "the question of the uniformity with which the bottom can be dredged".

In laying the 28-in. line, sufficient care was not taken to secure a uniform trench bottom, and, later, it was found that the pipe as laid varied in places as much as 2 ft. from the established grade. No harm resulted from this irregularity in the alignment, other than as it might affect the flow of water, for the flexible joints allowed the pipe to adapt itself to the ground surface.

With the 24 and 30-in. lines, greater care was taken to secure a uniform trench bottom, but it is doubtful whether it came within the 6-in. limit named by Mr. Wiggin. This would be extremely difficult to accomplish in a tidal stream, with a depth of water ranging from 20 to 40 ft., or more.

Mr. Wiggin states that "The exact method of making up and caulking the joints is of great importance."

The joints used permitted the lead to be caulked in two rings, and this method was followed.

In the case of the 24-in. steel pipe, laid in 1898, the specifications provided for the following procedure in setting up and testing flexible joints:

"The balls and sockets will be put together in the shop and subjected to tests by first filling with water and afterward by hydrostatic pressure up to 250 lb. per sq. in. In putting the flexible joints together, the socket will be first put on its end, small flange down, and then the ball will be inserted and fairly raised on the projection of the socket, previously bored to the proper size and angle. Then a fibrous gasket will be inserted and driven to the bottom of the cavity, when the melted lead will be poured from as many places as may be necessary to insure a solid body without seams or blow-holes.

"After the setting of the lead, the ball will be raised to ascertain the perfection of the work, and if found deficient it is to be made over. If found perfect, the ball will be replaced and the caulking done in the usual manner; then another gasket inserted as shown, the main gasket put in its place and bolted, but not quite home, when again the melted lead is poured into the cavity. To insure the perfection of the work, this main flange will be firmly bolted home and the final caulking done. If it is deemed necessary to warm up the parts in order to secure perfect work, it shall be done, at the request of the inspector. If during the process of testing any defects shall be manifest, all such shall be made good, and if required new parts will be substituted in place of the defective ones, all at the contractor's expense. While under test the ball will be moved from side to side and thus ascertain if it fits perfectly in all positions when in place.

"To prevent end strains on the ball-and-socket joint while under hydrostatic pressure, the caps shall be held in place by rods reaching

Mr. Clarke.

from one to the other. The joints shall remain perfectly tight while the pressure of 250 lb. per sq. in. is being applied, and shall so continue until the pressure is entirely removed."

For the 30-in. steel pipe, laid in 1910, a similar process was followed, the specifications providing as follows:

"The ball shall be machined truly spherical and shall have a smooth surface when finished. All flanges shall be faced flat, the end flanges finished for gaskets. When assembled the two parts of sockets shall match evenly and the space for lead shall be practically true to ball. For making the lead joint, lead wool shall be used, and each part of socket shall have lead joints made as a separate operation with strands at joints between castings. When finished each complete connection shall be subjected to a hydrostatic pressure of 300 lb. per sq. in. after having the angle of surfaces of end flanges moved at least 10° from relative position occupied by them when lead joint was made, under which pressure there shall be no leaks."

From the foregoing, it will be seen that provision was made for two rings of caulked lead for each joint, and the work was done in this manner. The flexible joints, however, were not moved or deflected while being made up.

In the case of the 30-in. line, it was proposed at first to use lead wool, but this was subsequently abandoned.

A question is also asked as to the extent of under-water caulking required.

The provisions for making up and testing the joints were carried out, and, as a rule, no large amount of caulking of lead joints was required after the pipe had been laid in the trench, although the pipe was carefully gone over by a diver, employed by the City, before it had begun to be covered by the sand and silt carried by the current along the bed of the river.

With the 28-in. line, laid in 1894, a pull on the joints was made in the process of laying, as was described in the paper by Messrs. F. and A. S. Riffle.

With the 24 and 30-in. pipes, resort was not had to such a method of producing tight joints. Probably the balls were turned or machined more uniformly, but, at all events, the larger leaks found after the pipe was laid were easily caulked. It should be stated, however, that there were doubtless small leaks which were not located, due, in part, to their being covered before the final test was made.

Reference has already been made to the volume of leakage from these pipes. When the 28 and 24-in. lines were laid originally, a small force pump was used for the final test for leakage. This pump took suction from a vessel of known capacity and, in this way, the volume of water required to maintain the pressure at the desired standard was measured.

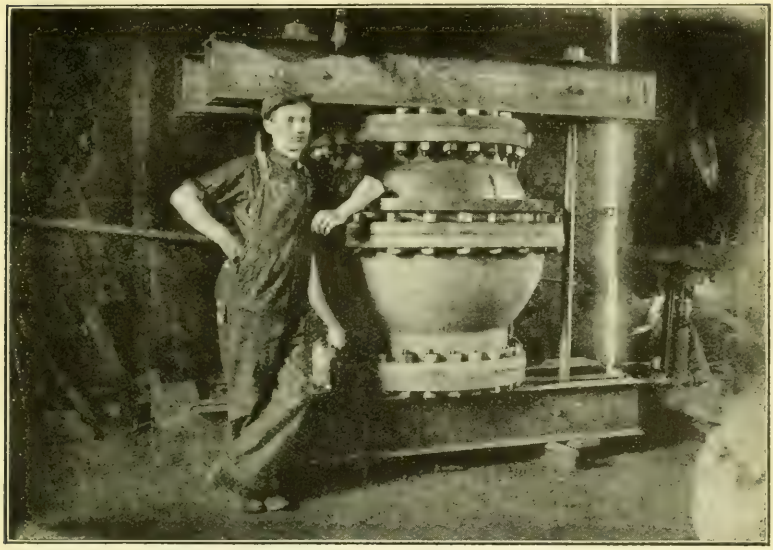


FIG. 9.—TEST OF 30-INCH FLEXIBLE JOINT, PORTLAND, ORE.

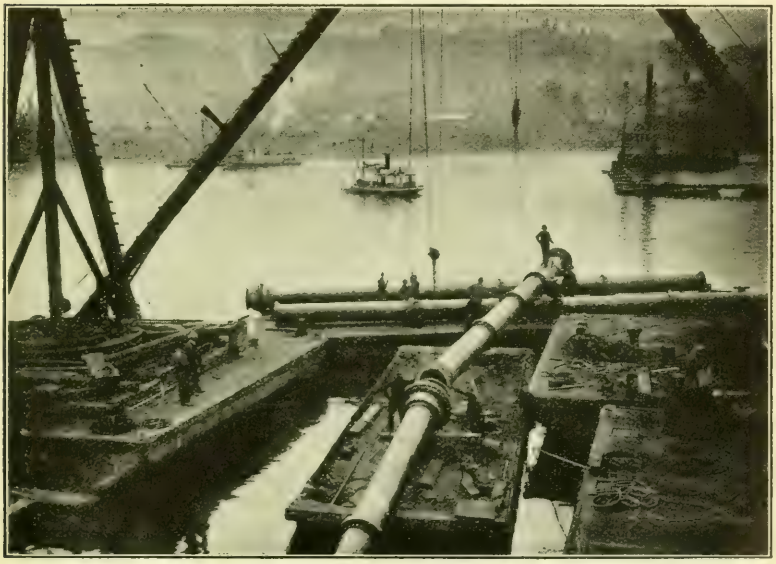


FIG. 10.—DERRICKS USED FOR LAYING 30-INCH PIPE, PORTLAND, ORE.

Mr. Clarke.

In later tests, made when two lines were in use, a small by-pass was inserted between the two mains, and a disk meter was installed to register the flow from the main in service to the one being tested, and in this manner the quantity required to maintain normal pressure in the main being tested was determined.

In the case of the 24-in. line, when first laid, the leakage approximated 10 080 gal. per day, the distance between the East Side and West Side gate-chambers, where stop-valves were set, being 2 706 ft., comprising 2 042 ft. of 24-in., flexible-joint, steel pipe and 664 ft. of 32-in., standard, cast-iron pipe laid on shore above water level.

For the 30-in. line, laid in 1910, comprising 2 265.6 lin. ft. of standand 30-in., cast-iron main and 2 047 lin. ft. of 30-in., lap-welded, steel pipe, with twenty-seven flexible joints (or a total distance of 4 312.6 lin. ft. between gate-valves), leakage tests were conducted in the manner already described, the quantity of water required to be pumped into the main, in order to maintain the desired pressure, being measured by meter. In this case, the first test did not result as favorably as was expected, the total leakage, under the normal pressure of approximately 150 lb., being about 30 000 gal. per day. Later tests, however, showed the leakage to be 5 000 gal. per day. After the 24-in. line was taken up and relaid, the test indicated a leakage of 2 100 gal. per day, which is regarded as a favorable showing.

Regarding the attempts which were made to reduce the volume of leakage by recaulking the joints under water, it may be stated that, after the pipe had been laid in the trench, and while it was under pressure, a diver examined all the joints thoroughly and recaulked them when necessary. This examination was made, for the most part, before the silting up of the trench had begun, but, at some points, it was not possible to do this before the pipe had been partly covered by the fine silt moving along the bed of the stream or breaking down from adjacent banks.

Mr. Wiggin raises an interesting question regarding the depth at which lead joints are affected by caulking, and also as to the effect which the pulling of the joints may have on their permanent tightness. The 28-in. line, laid under conditions which seemed to demand a pulling of the joints, has now been removed, and it is not possible to secure additional data regarding its water-tightness and the effect of the treatment to which it was subjected. The 30 and 24-in. lines were not handled in this manner, but the writer does not know of any way to determine how the joints are affected by temperature changes. On the assumption that the leakage would be greatest during warm weather, when there might be a slight expansion of the pipe due to the higher temperature, a test of the leakage can be made and compared with a similar test made in the winter, but the writer is doubtful as to the

Mr. Clarke.

results which might be obtained. The variation in the temperature of Bull Run water ranges from a minimum of, say, 32° in the coldest weather to an average of from 58° to 62° at the City's reservoirs in the summer, as it flows from the conduit.

The suggestion that meters should have been placed at each end of the submerged pipe lines is worthy of note. In fact, that is what has been done with the new conduit, No. 2, built in 1910-11, leading from the head-works to Mt. Tabor. A Venturi meter was placed at each end of this conduit for the purpose of determining whether there is any variation in the flow. A leaky joint would hardly be discovered in this way, but a large break would be detected at once.

The fracture in the 28-in. pipe is well described by Mr. Wiggin as having taken a long slant, "such as would occur in a coarse-grained stick of wood", and the most probable explanation of the break is that the metal of the pipe was resting on a hard spot in the pipe trench.

It may be said that, in laying the 24-in. line, rock was encountered at this point, and, in order to avoid blasting in close proximity to the 28-in. line, it was necessary to lay the second pipe close to the first—in the same trench, in fact. Therefore, it is considered more than probable that some projecting rock in the bottom of the trench, together with a heavy overlying embankment, caused the fracture of the 28-in., cast-iron pipe, which has been described. Fig. 11 gives an idea of the appearance of the break.

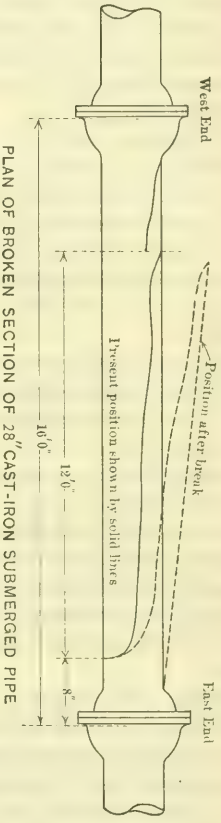
It has already been explained that when the pipe was uncovered, the two fractured pieces were found to be separated. The drawing of the two ends sufficiently close together to permit of their being enclosed in a steel sleeve did not prove to be an easy matter, and was not accomplished in an entirely satisfactory manner. The results proved that the wrapping of tarred canvas was not effective in preventing the cement grouting from finding its way through the joints in the layers of canvas and into the interior of the pipe.

When the 28-in. pipe was taken up, it was found that the cement had been deposited on the bottom of the pipe to the depth of several inches, but, fortunately, it was not in sufficient quantity to restrict the flow of water appreciably.

Concerning the purpose of the grouting and the benefit derived therefrom, it may be said that it was probably not worth its cost. The main thought was that the cement might aid in the caulking of the joints of the sleeve, and would also encase the pipe and hold it in position.

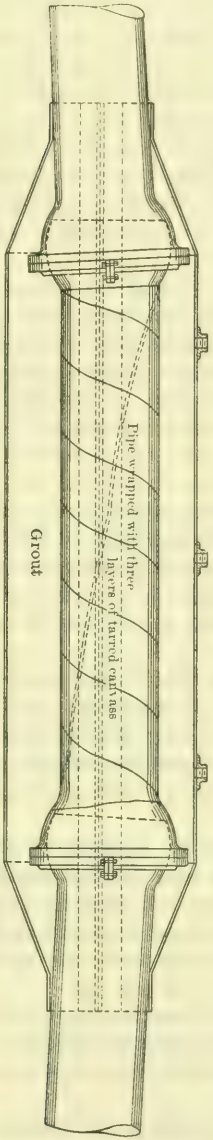
As has been stated, lead wool was used in caulking the circular joints of the sleeve around the pipe, but the longitudinal seams were packed with a rubber gasket. The probability that the grouting would be so impervious as to reduce the pressure on the flanges of the sleeve was remote, but it was one of the things which could be done at small

Mr. Clarke.



PLAN OF BROKEN SECTION OF 28" CAST-IRON SUBMERGED PIPE

FIG. 11.



REPAIR OF 28" CAST-IRON SUBMERGED PIPE AT PORTLAND, ORE. SHOWING SPLIT SLEEVE IN PLACE

FIG. 12.

Mr. Clarke.

expense and might help a little, one of the "straws"—so to speak—which a man will seek to grasp when in a difficult situation.

It should be noted that on one occasion in 1912 about two dozen of the flange bolts gave way ($1\frac{1}{4}$-in. bolts, 3 in. from center to center), owing to the heavy pressure to which the pipe was subjected. The broken bolts were replaced with bolts of Norway iron, and there was no more trouble while the pipe was in use.

Except at one place, near the West Harbor line, no attempt was made to back-fill the pipe trenches after the pipes had been laid, and there a small quantity of gravel was placed around and over the pipe where it came near the surface of the river bed in its upward slope toward the bank.

In a few places the diver found it necessary to support a joint or two on blocking, in order to avoid too sharp an angle, but, as a rule, the pipes, after being laid on the trench bottom, were left to be covered by the silt carried by the first freshet, the depth of the pipe below the river bed being deemed an ample protection. It should be said, however, that vessels are not allowed to use that portion of the harbor as an anchorage ground, the river being also too narrow for such use by larger vessels.

Mr. Wiggin's inquiry regarding the shop methods used in the manufacture of the flexible joints has been answered by Mr. Phillips, as observed by him in the manufacture of the joints for the 30-in. pipe.

The inquiry of Mr. Spear regarding the accuracy of the final grade and alignment for the submerged pipes has already been touched on in reply to Mr. Wiggin.

Regarding the condition of the 28-in., cast-iron, ball-and-socket pipe taken up after 20 years' service, it may be said that the pipe itself, in the main, is in perfect condition, but slightly rusty. Before relaying, it will be necessary to turn down the balls slightly merely to give them a fresh and uniform surface, no pitting of any depth having taken place.

Answering Mr. Hogan's request for information regarding the length of time required to make up and lay the flexible joints: Mr. Randlett notes that the length of time required for laying the underwater portion of the 30-in. line (an approximate length of 1 600 ft.) was only 9 days, or, say, 180 ft. per day; but this would involve the laying of only three of the flexible joints per day.

No attempt was made to smooth down the bed of the trench after the dredge had finished its work, the flexible joints being depended on to take care of such slight inequalities of surface as would be likely to occur.

The question raised by Mr. Brush as to the accuracy with which pipes can be laid under water, or "how much deviation in line and grade is permitted in submerged pipe construction?", is of interest,

but is also difficult to answer. In theory, the pipe should conform to the designated lines and grades; but, in practice, "it is different". In answer, therefore, the writer can only describe his experience with the several pipe-line crossings laid in Portland. Mr. Clarke.

As has been stated, the 28-in., cast-iron, flexible-joint pipe, built in 1894, was laid in a trench excavated with a ladder-dredge which was supposed to excavate to a uniform depth below the surface of the water, the ladder being adjusted according to the gauge showing the variation of water levels from the low-water line, which might occur from time to time. For laying the pipe, a timber cradle was constructed, one end of which rested on the pipe-laying barge and the other on the ground in the bottom of the excavated trench. This cradle was pulled from under the pipe as the pipe-laying barge was advanced, thus depositing the pipe in the trench along the line which the cradle followed, but it might or might not be exactly the alignment intended. That this pipe was not laid exactly to either line or grade as originally designed was shown by a survey made in 1902, when the question of lowering the pipe was under consideration. Soundings made at that time showed variations or depressions in grade amounting in places to as much as 2 ft., although no abrupt changes were noted.

In laying the 24-in. pipe, built in 1898, the same pipe-laying methods were adopted as had been used 4 years before, but, on account of the pipe being somewhat lighter, it is probable that a better alignment was secured.

In laying the 30-in. pipe, in 1910, and in relaying the 24-in. pipe, in 1913, a different method was adopted, the pipe being suspended from three boom derricks, and it was thus possible to line up the cables supporting the pipe and keep them in line until it was lowered to the bottom of the trench. By such means, the writer should say it is possible to deposit a pipe within a foot of the desired alignment, if the depth of water is not excessive. Any closer work than this he would not expect.

Mr. Ward inquires regarding the use of lead wool in caulking the flexible joints. The specifications for the 30-in. line provided for the use of lead wool, but, after a trial, its use for such purpose was abandoned. One reason for its disuse was the danger of marring the turned face of the ball by the excessive caulking required to consolidate the lead.

The writer feels under special obligations to Messrs. Clemens Herschel, L. J. Le Conte, Kenneth Allen, and R. C. Kellogg, as well as to the gentlemen whose questions he has endeavord to answer, for their interesting contributions to the discussion and for suggested improvements in laying flexible-joint pipe.

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PAPERS AND DISCUSSIONS

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HISTORY OF LITTLE ROCK JUNCTION RAILWAY BRIDGE, OF THE ST. LOUIS, IRON MOUNTAIN AND SOUTHERN RAILWAY COMPANY, OVER THE ARKANSAS RIVER AT LITTLE ROCK, ARKANSAS, 1883-1914

Discussion.*

By C. E. Smith, M. Am. Soc. C. E.

Mr. Smith.

C. E. Smith,† M. Am. Soc. C. E. (by letter).—Mr. Quimby mentions a very interesting case in which considerable settlement took place in a timber crib about two-thirds as high as the Little Rock crib and of much better construction. Had the Little Rock cribs been constructed similar to those described by Mr. Quimby, no apprehension would have been felt, regardless of the settlement.

The writer was very much surprised and filled with regret to learn that any one thought the paper reflected on the ability of any of those able engineers who had to do with the construction of the bridge or the correction of the trouble, as no reflection or criticism was intended, the paper having been written merely as a recital of facts.

Fortunately, full detailed information in the shape of original papers was available, and quotations were freely used where available to render the subject matter more accurate and to avoid any misunderstanding that might have arisen through digests of the quoted matter.

* Author's closure of the paper by C. E. Smith, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† St. Louis, Mo.

Mr. Smith.

The writer believes that, with the knowledge at hand during the construction of the bridge and during the trying period of repairs, all those who had any connection whatever with the work handled the matter according to their best judgment, in conformity with the facts developed and resources available. Unfortunately, funds for better construction and for the proper correction of the trouble were not at all times available, but had they been there is no doubt that any one of the large number of able engineers connected with this problem would have corrected the difficulty in an entirely efficient and satisfactory manner long before the writer had any connection with it.

This is definitely shown in the following quotation from Mr. Purdon's letter in the original report, which indicates that he objected strenuously to the type of construction:

"I objected to this strongly at the time, but as I did not design the piers, this work being done by the late Mr. T. E. Sickels, I was not responsible for them. I told Mr. Wood at the time that I was satisfied the timbers would eventually crush and put the piers out of shape, which it seems occurred."

In closing, the writer desires to express his appreciation to those who helped him in the final solution of this problem and who contributed so largely to the information now available; he wishes to express his regrets and apologies to any one who may have felt injured in any way by the paper as presented.

The association of the writer with all who have been connected with the work has been a source of great pleasure and benefit to him, and he has appreciated to the fullest extent the efficiency and integrity of those able engineers who struggled so faithfully with this problem, without the funds for proper solution.

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DISCUSSION ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE*

By W. M. Kinney, Jun. Am. Soc. C. E.

W. M. Kinney,† Jun. Am. Soc. C. E.—In considering the definitions in this report, it must be remembered that although, at this time, the report covers only bituminous materials for road construction, the work of the Committee has been broadened to include non-bituminous materials, and, unless consideration is given to this aspect of the report, greater confusion is likely to arise than if no definitions whatever were furnished. It is hardly conceivable that the Committee will wish to have different definitions for the same words used in two general types of construction, and it would seem preferable to have them worded so that they will apply to either type. In general, the instructions can be criticized in that they encroach on the province of the specification, being in too much detail to be used as definitions for all types of road construction. Mr. Kinney.

The speaker's objections to the individual definitions follow, and are based on the two questions:

(1) Will the definition apply to all types of road construction?
(2) Is the definition too specific for general use?

Aggregate.—It is customary to specify that aggregate be inert material, and a better wording of this definition might be as follows:

"Inert mineral material, such as sand, gravel, slag, or broken stone, or combinations of one or more of these materials used with cement or bituminous material to form a mortar or concrete."

* Discussion of the Progress Report of the Special Committee on Materials for Road Construction for 1914, continued from April, 1915, *Proceedings*.

† Chicago, Ill.

Mr. Kinney.

This definition should be amplified by sub-divisions defining both fine and coarse aggregate. In such case the following is suggested:

"Aggregate.—Fine.—Inert mineral material, such as natural sand or stone screenings, which will pass a screen having 4 meshes per linear inch.

"Aggregate.—Coarse.—Inert mineral material which will be retained on a screen having 4 meshes per linear inch and consisting of particles which generally do not exceed 2 in. in greatest dimension."

Cement.—The use of the term "adhesive substance" is hardly correct. This definition could be made more general, such as: "A material possessing the property (under suitable treatment) of uniting particles of aggregate or bonding one material to another."

This definition should be sub-divided, and additional definitions should be furnished, as follows:

"Bituminous Cement.—A bituminous material suitable for use as a binder having cementing qualities which are dependent mainly on its bituminous character."*

"Portland Cement.—A cement obtained by finely pulverizing clinker produced by calcining, to incipient fusion, an intimate mixture of properly proportioned argillaceous and calcareous substances, with only such additions subsequent to calcining as may be necessary to control certain properties. Such additions shall not exceed 3%, by weight, of the calcined product."†

"Natural Cement.—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas."‡

"Puzzolan Cement.—An intimate mixture obtained by finely pulverizing together, granulated basic blast furnace slag or volcanic ash and slacked lime."

Cement-Concrete.—When taken in conjunction with the definition for cement, this term will be ambiguous unless one of the words, "Bituminous", "Portland", "Natural", or "Puzzolan", is prefixed. It would be best to separate and define each of the concretes as follows:

"Bituminous Cement-Concrete.—An intimate combination of bituminous cement and aggregate incorporated together by mixing methods.

"Portland Cement-Concrete.—A composite mass formed by intimately combining suitable proportions of coarse and fine aggregate, Portland cement, and water, and allowing the mixture to harden.

"Natural Cement-Concrete.—A composite mass formed by intimately combining suitable proportions of coarse and fine aggregate, natural cement, and water, and allowing the mixture to harden.

"Puzzolan Cement-Concrete.—A composite mass formed by intimately combining suitable proportions of coarse and fine aggregate, puzzolan cement, and water, and allowing the mixture to harden."

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* The definition on page 3011, *Proceedings*, Am. Soc. C. E., for December, 1914.

† *Circular No. 33*, U. S. Bureau of Standards.

‡ *Manual*, Am. Ry. Eng. Assoc., 1911.

Mr. Kinney.

In the foregoing, it will be noted that fine and coarse aggregate are substituted for the terms, gravel, shell, slag, sand, etc., inasmuch as fine and coarse aggregate have been previously specified as being made up of such materials.

Cement-Concrete Pavement.—This should be changed to Portland-Cement-Concrete Pavement, as Portland cement is the only cement, other than bituminous, used for road surfaces. Bituminous concrete pavement is elsewhere defined. The following definition could be used:

"Portland-Cement-Concrete Pavement.—One in which the wearing course, as well as the base, is composed of Portland-cement concrete. Frequently the base and wearing course are homogeneous."

As distinguishing between the bituminous-concrete pavement and the Portland-cement-concrete pavement having a thin wearing surface of bitumen and sand or stone chips, it might be well to term such pavements "Bituminated-Concrete Pavements". A suitable definition for such pavements would be:

"Bituminated-Concrete Pavement.—A Portland-cement concrete pavement on which has been applied at the time of construction, or after the pavement has been subjected to some wear, a thin carpet of bitumen and coarse sand, or stone, or slag chips."

Clay.—It is hard to justify the 200-mesh sieve dividing line given in the report, as well as the dividing lines between sand and clay, and gravel and sand given elsewhere. The quantity of clay in a sample of sand is seldom determined with a sieve, but most frequently with a suitably designed apparatus by passing water at a certain pressure up through a certain known quantity of material, by which water the clay or silt is carried off in suspension. Arbitrarily using the 200-mesh sieve as the dividing line between clay and sand is incorrect, as sand containing no clay whatever frequently has material passing the 200-mesh sieve. The refinement used in the report is outside the scope of a definition. The following is suggested:

"Finely divided earth, generally of silicious and aluminous nature, and usually in the form of an impalpable powder which will remain in suspension in water for a varying length of time, depending on the fineness of the particles."

Consistency.—This term is also used in referring to concrete mixtures, and an addition to the definition should be made as follows: "The degree of firmness or fluidity of cement mortar or concrete."

Crusher-Run Stone.—As defined in the Committee's report, this should be termed "Coarse Aggregate". Crusher-run stone is everywhere understood to be the product of the crusher, without the removal of any fine material. A distinction between "Crusher-Run" and "Crusher-Run Stone" is an unnecessary refinement.

Mr. Kinney.

Crust.—The speaker does not see the necessity for using, and hence for defining, this word, in connection with road materials in place. The Committee's definition of "Crust" conflicts with the definitions for "Wearing Coat" and "Wearing Course". "Crust" may properly refer to a covering of definite thickness, such as a baked surface on natural soil. The word as used by the Committee makes it include the entire macadam construction; in other words, macadam and crust are made synonymous. (A reference to "Wearing Coat" and "Wearing Course" will show the inconsistency of the definitions.)

Dust.—This definition might be simplified as follows: "Earth, pulverized stone, or other matter in the form of a fine, dry, impalpable powder." In this manner the definition includes crusher dust.

Foundation.—In the Committee's definition, crust and pavement are again made synonymous. Mention of sub-base should be included in some manner after that in the suggested definition, and sub-base may be further defined as "suitable material placed on the properly prepared sub-grade to assist in forming a suitable foundation".

Sub-base would seem to be a better term than "Artificial Foundation".

Gravel.—The basis for differentiation between gravel, sand, silt, and clay is not in accord with accepted practice. Nowhere, in practice, to the speaker's knowledge, is the 10-mesh screen used for separating sand and gravel. In all concrete work the dividing line is the 4-mesh screen or $\frac{1}{4}$-in. holes.*

It is true that in some work, such as grouting brick pavements and plastering, a finer sand (passing a No. 8 screen) is required, but this necessity need not alter the low limit set for gravel. As sands for concrete are usually deficient in coarse particles, any material between $\frac{1}{8}$ and $\frac{1}{4}$ in., obtained from the production of grouting or plaster sands, may be added to concrete sands. As indicated under the discussion of "Clay", arbitrarily setting the dividing line between sand and clay or silt at the 200-mesh screen is incorrect, as the determination is not made in that manner. A definition for "Gravel", meeting all conditions of use, would be as follows:

"Properly graded, screened material from a natural deposit, consisting of small stones or pebbles ranging in size from those that are retained on a screen having 4 meshes to the linear inch and including particles which generally do not exceed 2 in. in greatest dimension."

It would also be well to define "Bank Run Gravel" as "A natural unscreened mixture of sand and gravel containing usually some impurities, such as silt or clay".

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* Report of Special Committee on Concrete and Reinforced Concrete, *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 406.

Mr. Kinney.

Pea Gravel might be defined more technically as "screenings from sand or bank run gravel, usually ranging in size from about ⅛ to ½ in.". The term "Grits" is more common than "Grit", and with the foregoing definition in mind, "Grits" may be defined as "Screenings from crusher-run stone or slag, usually ranging in size from ⅛ to ½ in.".

Mortar.—Mortars are sometimes made from stone screenings, and, unless it is advisable to define Portland cement mortar, natural cement mortar, etc., as in the case of concrete, the following definition could apply:

"Mortar.—An intimate mixture of fine aggregate and cement with sufficient water to render the combination plastic."

Normal Temperature.—The normal temperature at which Portland cement testing is done is 21° cent., or 70° Fahr.*

This makes such a definition inapplicable for use by this Committee, without other limitations.

Pavement.—With sub-grade, sub-base, foundation, base course, and wearing course defined, there is no reason why the natural definition for pavement should be other than "that portion of a street or road improvement between the curbs or shoulders, and above the sub-grade, on which traffic is to be carried".

Resurfacing.—This definition is not broad enough to cover removing and replacing the wearing course or placing a new wearing course on an old pavement.

Road Metal.—This is usually understood to mean the paved portion of a roadway between the shoulders. The term is used thus in defining "Shoulders".

Sand.—This definition is already discussed under "Gravel", and should be defined as "inert mineral material, relatively free from clay, silt, or loam, occurring in, or screened from, a natural deposit, and passing a 4-mesh screen".

Screenings.—It is more common to make the dividing line between screenings and crushed stone the 4-mesh screen.† Most crushing plants are now thus equipped.

Silt.—As silt frequently occurs as a coating on gravel, and would not be removed by sieving, even after drying the sample, the unsuitability of this definition is apparent.

Surfacing.—Portions (1) and (2) of the Committee's definition conflict with the definitions of "Crust" given elsewhere. The speaker thinks that these divisions of the Committee's definition should be

* Report of the Special Committee on Uniform Tests of Cement, *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 682.

† Report of the Special Committee on Concrete and Reinforced Concrete, *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 405.

Mr. Kinney.

eliminated, as they are not appropriate. The definition would then be as follows:

"Surfacing.—Finishing the surface of a roadway. Treating the surface of a finished roadway with some material which is to form the wearing surface proper."

Surface Treatment.—There are methods of surface treatment other than with bituminous materials, such as with calcium chloride, sodium silicate, etc.

Wearing Coat.—If the wearing coat is the superficial layer of the crust, then, according to the definition given elsewhere for the "Wearing Course", the entire pavement becomes a superficial layer. Wearing coat should be defined so that it will apply specifically to the second course of two-course concrete construction, or to the bituminous coating, or any similar surface intended to receive directly the wear of traffic.

"Wearing Coat" and "Wearing Course" are so nearly synonymous that there seems to be no reason for defining these terms separately.

In presenting these definitions, the Committee does not restrict their use to bituminous highway construction, but "recommends that the following lists of terms of frequent use in expressions relating to highway work be recognized as having the meanings set forth in the list". Apparently, sufficient thought has not been given to their applicability to other than bituminous construction.

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SUGGESTED CHANGES AND EXTENSION OF THE UNITED STATES WEATHER BUREAU SERVICE IN CALIFORNIA

REPORT OF A COMMITTEE OF THE SOUTHERN CALIFORNIA ASSOCIATION OF MEMBERS.

Discussion.*

BY WILLIAM S. POST, ASSOC. M. AM. SOC. C. E.

WILLIAM S. POST,† ASSOC. M. AM. SOC. C. E. (by letter).—There is one feature of the Weather Bureau Service which only has to be stated to be recognized, namely, that this Bureau, as a matter of fact, has a monopoly of publication. It is, of course, a national organization, and perhaps cannot recognize local conditions to too great an extent, but the writer wishes to emphasize publication as an important matter, in addition to the recommendations of the Committee. There are numerous rainfall records in California, and presumably in other States, which are in the hands of private parties and cannot qualify as official. These records are of the utmost importance because they frequently go back as far as the early Seventies, and are fast disappearing with the dying out of the original observers. We have obtained or are maintaining some 60 stations alone in San Diego County which have no official means of publication. The writer would suggest that the Weather Bureau consent to publish all records in the State, in a volume devoted simply to rainfall, to be contributed from all known sources of information, the responsibility for the record being stated. In this way an extremely important source of information will be preserved. Mr. Post.

In reference to this matter the writer, alone and single-handed, some two years ago, attempted to convince the head of the Weather

* Discussion of the paper by George S. Binckley, M. Am. Soc. C. E., and Charles H. Lee, Assoc. M. Am. Soc. C. E., continued from April, 1915, *Proceedings*.

† San Diego, Cal.

Mr. Post. Bureau that the form of publication, in so far as it gives long lists of "normals", is a needless incumbrance of the present reports. To state it differently, what is required is not the mathematical interpretation of differential variations, etc., but a plain tabulation of the observed data. The first result of this, to the great relief of some engineers, would be a publication, 8½ by 11 in., which is a standard filing size, and the increase of the size of the type from a microscopic nonpareil to a good book type. It may be possible to convince a physical scientist that these normals have some significance, but most hydraulic engineers will agree that they are an unmitigated nuisance in the present publications.

The writer has suggested to one of the members of the Committee that, in case it is not possible to co-operate fully with the Weather Bureau in this matter of publication, the Society itself or the Local Association make permanent the present committee and undertake the publication of the meteorology of California, combining as contributors all those who can furnish any data.

In hydraulic calculations, certain stations have become in the nature of base stations, and are very valuable for the purpose of restoring broken records or years of no record in the same region. For this reason it is a real loss to have a long-term station discontinued, merely, as it would seem, because the voluntary observer has moved and the Weather Bureau official has neither the money for traveling expenses to inspect and secure another observer, nor funds to hire a paid observer. The writer has recently expressed his regret at the discontinuance of the splendid record at Fort Wingate, in Arizona, to the extremely intelligent and competent Director of the Weather Bureau at Santa Fé, N. Mex., and was told that his hands were absolutely tied in the matter, that he could only secure a voluntary observer through correspondence, and had attempted to do so. That is a case where, because of the lack of an efficient field inspector, who could discover observers, a record which dates back to 1867 has been broken for about 3 years. Numerous other instances could be brought up.

The suggestion is also made that the Weather Bureau take up the study of local drainage areas for a few years, in order to determine the local shapes of the isohyetose lines once and for all in that area, and thereafter proceed with only one base station in that drainage area. The writer has succeeded in drawing such curves on certain drainage areas in Southern California by the establishment of some fifty rain gauges. In some cases there are voluntary observers and in others $1.00 per month is paid. About 15 gauges are examined by one hydrographer, who is also a good rider and can crank a machine. From a comparison of these curves, now that they have been established, it is obvious that a very considerable reduction in these stations can be made.

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CINDER CONCRETE FLOOR CONSTRUCTION BETWEEN STEEL BEAMS

Discussion.*

By A. L. A. Himmelwright, M. Am. Soc. C. E.

A. L. A. Himmelwright,† M. Am. Soc. C. E. (by letter).—This paper reviews briefly the history of cinder concrete fire-proof construction and tests in New York City. Mr. Himmelwright.

Having been identified with the early development and investigations of fire-proofing, and particularly cinder concrete floors, the writer found much of interest in the paper. Many of the comments and criticisms of the authors in regard to the early investigations are justified and sound. In some cases, however, they have fallen into the same errors that proved to be stumbling blocks to some of the early investigators. As in the paper‡ by Guy B. Waite, M. Am. Soc. C. E., the investigation seems to have been conducted solely along the well-beaten path of dense concrete mixtures and conventional methods of reinforcement. The title, "Cinder Concrete Floor Construction Between Steel Beams" was broad enough to lead to the expectation that other forms of floor construction than the conventional reinforced slabs might be included; but in this respect the paper has proved disappointing.

Notwithstanding all that has been written on fire-proof floor construction during recent years, the writer is thoroughly convinced that a large field of this subject has been practically unexplored, and in this, sooner or later, will be found a more efficient and economical method of floor construction than any of the conventional methods now in use.

* Discussion of the paper by Harold Perrine and George E. Strehan, Juniors, Am. Soc. C. E., continued from April, 1915, *Proceedings*.

† New York City.

‡ "Cinder Concrete Floors," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1773.

Mr. Himmelwright.

In reinforced concrete slabs, the metal reinforcement is placed on the tension side of the slab and becomes more and more efficient in developing strength as it approaches the under surface of the slab and as its distance increases from the theoretical neutral axis. The nearer it approaches the under surface of the floor slab, however, the more it becomes exposed to heat in case of fire, and the sooner it becomes weakened by the effect of heat. This is a fundamental principle of weakness, which is present in every type of reinforced concrete construction.

Where a double construction (a floor and a separate ceiling with a space between) is desirable, as in office buildings, etc., there is no particular advantage in having a horizontal surface on the under side of the floor. In the case of such buildings, therefore, a segmental arch type of floor could readily be used, and as this type is entirely independent of metal reinforcing elements, and the entire section of the arch material is in compression, it is a much superior fire-resisting type, and if cheap methods of centering and erection were developed, it would undoubtedly be a less expensive and more efficient type of flooring than any of the flat reinforced floor constructions now in general use. A segmental arch, of a uniform thickness of, say, 3 or 3½ in. of very porous material, would afford ample strength, and would be much lighter and more economical than the flat slabs ordinarily used. A porous concrete such as is here suggested, weighs only about 7 lb. per sq. ft. per inch of thickness, and when erected on open centering, which does not retain the moisture, such a concrete can be made in any temperature without seriously impairing its strength or other qualities.

Conclusive tests by the writer, 18 years ago, and witnessed by a number of reputable engineers and builders, thoroughly established the fact that porous cinder concrete at a temperature of 38 to 40° can be placed on mesh or similar open centering in the usual manner in temperatures as low as 10° below zero without materially affecting the concrete. This is a commercial and practical advantage of enormous value in large cities where fire-proofing is frequently required to be built during the winter.

With the foregoing brief outline as to the possibilities of a porous concrete in segmental arch form, it is more or less astonishing that no progressive firm has given the attention and thought to this method of construction, which its commercial possibilities deserve.

Reverting now to the limited subject of the paper, the discussion will be treated under the headings into which the paper is divided.

Fire Resistance.—Considerable space is devoted to comment on what is termed the "pitting" on the surface of the concrete, which is stated to be "due chiefly to the combustion of small particles of exposed unburned coal, in addition to the effect of de-hydration". The state-

ment is also made, that "one of the chief arguments, derogatory to cinder concrete as a fire-resisting material, is that unburned coal, occurring in the aggregate in appreciable quantities, when burned out, will produce flaws and honeycombing, and consequent loss in strength". Several illustrations are presented to show this "pitting" and "honeycombing" effect. Mr. Himmelwright.

It would be interesting to know just how the quantity of unburned coal was determined in Table 5 giving the chemical constituents of cinders. The table states "carbon loss on ignition". If this was ascertained by heating to a high temperature, it is easy to account for the large percentage of "unburned coal" given in that table, running all the way from 11 to 28% of the total cinder.

Tests of the black particles which resemble unburned coal, made at the suggestion of the writer, showed that less than 7% of these particles would ignite at the temperature at which ordinary coal of the same size would be consumed and leave an ash residue. These tests proved that the black particles occurring in cinder are composed very largely of impurities, and their presence is thus explained, after passing through the furnaces of the steam boilers, etc., in which the coal was used.

It was also found, in the tests referred to, that, by raising the temperature, additional quantities of the black particles were consumed. It is well known, of course, that rock, slate, and similar foreign matter will disintegrate under the action of heat, and when exposed on the surface of the concrete to sudden cooling by water, will readily separate from the concrete and leave voids. It is the writer's contention, therefore, that only a part, and relatively a small part, of the pitting is due to unburned coal.

It would have been interesting if some tests or other data had been available to compare the fire resistance of dense or full concrete, with concrete of greater or less porosity. The writer's experience indicates that a full or dense concrete, in which practically all the voids are filled, is much more easily cracked and damaged by heat than a porous concrete; similarly, a stone concrete, with the same proportion of voids, cracks more readily on the application of heat than a cinder concrete.

In their behavior, the dense or full concretes resemble more nearly the different natural stones of which the aggregates are composed, in proportion as they are dense and full. In the case of cinder concrete, it is impossible to attain complete density, on account of the character of the cinder aggregate, which, in itself, contains considerable interior voids which cannot be filled by the cementing material, even when thoroughly tamped. It would be a comparatively simple matter to make tests of different degrees of porosity in cinder concrete, and show its relationship to fire and frost resistance. This would be

Mr. Himmelwright.

valuable information in the development of more economic methods of construction. Lightness is highly important in economic designing, as it materially reduces the dead weight of the floors. This feature of concrete floor construction seems to have been wholly lost sight of in recent years. The reason of this neglect is undoubtedly due to incomplete investigations and the mistaken conclusions in regard to the corrosion of metal in cinder concrete.

Corrosion.—Under this heading is found the unwarranted recommendation of the Structural Association of San Francisco, which the writer condemned at the time it was made, shortly after the San Francisco fire, as being entirely unjustified by the actual phenomena presented by that conflagration.

In the past, and particularly in the case of work executed more than 8 or 10 years ago, reinforcing metal for floor construction was never protected against corrosion before being placed in the work. As a consequence, such metal was often oxidized over its entire surface, and was used in this condition. This was generally characteristic of Western practice, the writer having in mind the fire-proof floor work in San Francisco, Los Angeles, and other Western cities. In investigations made by the Structural Association of San Francisco, and by individuals, this fact was evidently not taken into consideration, and undoubtedly these are some of the "extreme and unexplained causes of corrosion" which have been pointed out from time to time.

In the examination made by the writer immediately after the San Francisco fire in 1906, very little of the exposed reinforcing metal was found to be free from corrosion, even where the concrete had been in contact with the metal. All these cases of oxidation were undoubtedly due to exposure to moisture in the salt air of the Pacific Coast before being placed in the work.

The conclusions of Professor Norton, of the Insurance Engineering Experiment Station, are also reviewed. These conclusions were discussed by the writer* some years ago, when they were first published, and the criticism made was that the investigations were not carried far enough, and that they were erroneously applied to structural steel.

These conclusions are based wholly on experiments with naked or unprotected metal embedded in the concrete. It is well known that structural steel, as furnished in practice, is usually protected with two or more coats of a suitable weather-proof paint, and this has an important bearing on the problem of corrosion, in so far as structural steel is concerned. Professor Norton's conclusions, and the writer's own investigations, have clearly proved that naked steel will be attacked opposite voids in the concrete, or where the cement is not in intimate contact with the metal surface. This fact, *per se,* widely

* In *Engineering News,* and in other technical publications.

Mr. Himmelwright.

published and advertised, has had the effect of restricting all effort, in the development of concrete floor construction, to the use of dense or full concretes, and, because of this unfortunate result, Professor Norton's findings, though correct as far as they go, have actually done more harm than good in the advancement of economic methods of cinder concrete floor construction. Had Professor Norton continued his investigations and ascertained that, in the absence of moisture, oxidation adjacent to voids progressed very slowly and could practically be neglected—which is a truth as important as the other—much of the harmful consequences of his conclusions would have been avoided.

When the writer first engaged in the business of putting in fire-proof floors, in 1896, cinder concrete was a new material; in fact, as far as he knows, his firm was the first to recognize its merits and use it for that purpose on a large scale. It was desirable to ascertain definitely whether or not it was suitable for fire-proof floor construction, and, more important than anything else, whether or not it had any harmful effect on the steel with which it came in contact, or any other undesirable properties for the purpose intended. As this involved a very thorough chemical investigation, the writer engaged Messrs. Booth, Garrett, and Blair, well-known chemists, of Philadelphia, Pa., to make the investigation as to the corrosive effects of cinder concrete. This investigation and report was made more than 18 years ago, and was very extensively distributed among the architects, engineers, and building contractors of New York City at that time. With the full information, as given by this report, and with all the information of nearly 20 years' experience available since that time, it seems incredible that the authors should make the statement that "the corrosion of steel embedded in cinder concrete is still a matter of conjecture". The detailed investigation of Messrs. Booth, Garrett, and Blair was so thorough and replete with information that the writer feels that it would not be amiss to reproduce it in its entirety, together with the supplementary tests and investigation suggested by the preliminary report.

"REPORT ON THE EFFECTS PRODUCED BY CINDER CONCRETE ON STRUCTURAL STEEL IN CONTACT WITH IT, OR EMBEDDED IN IT.

"March 3, 1898.

"MESSRS. JOHN A. ROEBLING'S SONS CO.,

117 Liberty Street, New York.

"GENTLEMEN.—Agreeably with the instructions received from Mr. Himmelwright of your company, we have had under consideration for some six weeks past the question stated in the heading, to wit, the effect of cinder concrete as used by you in floor arches, etc., upon steel embedded in it; and are now prepared to report on the matter as set forth in detail below:

Mr. Himmelwright.

"On January 7, we received from you, by express, samples, as follows:

"*Samples.*—1, Cement; 2, mortar sand; 3, anthracite cinder.

"We understand these samples to represent, respectively, one, Aalborg Portland cement; two, Cow Bay sand; and three, cinder from boilers of the New York Steam Co., using anthracite pea coal. We further understand that the proportions in which these ingredients are used in concrete are, one cement, two sand, and five cinder.

"*Corrosion of Iron and Steel.*—In dealing with corrosion of iron and steel, three active agents producing it are recognized. These are water, carbonic acid, and sulphuric acid. Pure water, free from carbonic acid, will not attack iron, but unprotected ironwork is rapidly corroded by either carbonic or sulphuric acid in aqueous solution or by contact with these acids in a damp atmosphere. It is, of course, a fact that in structural work the metal is generally protected by a covering of paint of some sort or other. Such coatings, however, vary greatly in the efficiency with which they protect the iron; many of them are destroyed themselves by the presence of either acids or alkalies, and all of them are likely to be removed in places by abrasion or other mechanical effect. Hence, for the purposes of this investigation, the coating on the iron may be dismissed as irrelevant, and the question then becomes, what corrosive effect a concrete of one cement, two sand, and five cinder, as per samples received, would exert on unprotected structural steel? Again, since both carbonic acid and moisture are always present in the atmosphere, it may be assumed that any corrosive effect which they can produce would be exerted in any case as conditions favored. It is not therefore necessary to make any investigation to discover the presence of carbonic acid; it is only necessary to consider whether it could produce corrosion on iron embedded in this concrete. The investigation thus narrows down at the outset to determining the presence of sulphides and sulphates in the concrete.

"There is shown by analysis of the samples received:

"*Sulphates and Sulphides in Concrete.*—

	Sulphuric Acid.	Sulphur as Sulphide.
"Cement	2.083 per cent.	Trace.
Sand	None.	Trace.
Cinder	0.213 per cent.	0.188 per cent.

"The figures for sulphuric acid and sulphur in the cinder are lower than we expected and lower than is generally supposed. On the supposition that the sulphur might be found in fine ashes instead of in the cinder, we requested you to send for another sample to be taken directly from the chute. Analyzing the fine ashes in this, we obtained:

"*Sulphates and Sulphides in Ashes.*—

"Sulphuric acid	0.161 per cent.
Sulphur as sulphides	0.338 " "

"This shows a little more. It occurred to us that the sprinkling of the ashes to cool them might dissolve out some of the sulphur, so we asked for a third sample to be taken directly from the ash pit.

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"This analyzed:

"Sulphuric acid (SO_3)	0.144 per cent.
Sulphur as sulphides	0.116 " "

"This is the lowest determination of the lot. The conclusion we reach is that the sulphuric acid in the cinder may be reckoned at 0.20%, and the sulphur as sulphides at about the same figure—0.20 per cent. The sulphuric acid present in the samples is, of course, in combination—in the cement, as sulphate of lime, and in the cinder, probably also as the same salt.

"The sulphides in the cinder are, however, capable of reacting with water to form sulphate, and if not combined with or surrounded by lime, are capable of corroding the iron.

"Effect of Sulphuric Acid.—The determination of the sulphuric acid present is important because of the remarkable effects produced upon the cement by this salt. These effects are two-fold, *viz.*, one, to modify the setting time; and two, to modify the tensile strength.

* * * * * *

"Without stopping to discuss the reasons of these phenomena (which are fully confirmed by numerous authorities), it is sufficient to point out that sulphate of lime in quite small quantities acts to retard the setting of cement and to increase its strength. In larger quantities, it is injurious, reducing the strength of the cement.

"Total Sulphuric Acid.—Reckoning the total sulphuric acid in the concrete to determine its proportion to the cement, we have:

"100 parts cement, sulphuric acid (say)			2.10 per cent.
200 " sand,	"	" (none)	
500 " cinder (0.20 SO_3)			1.00 " "
	(0.20 S hydrated SO_3)		2.50 " "
			5.60 " "

or the total sulphuric acid present in the concrete would be 5.6% of the weight of the cement.

"Fractional Part of Acid Only Effective.—If this amount of sulphuric acid were actually able to react upon the cement, it would very seriously affect it; and, if we suppose the cinder to be ground to an extremely fine powder and intimately mixed with the cement before hydration, then some such effect would no doubt be produced. But as the sulphuric acid is actually disseminated through a mass of cinder in lumps, say ½ in. in diameter, many of which are hard and semi-vitrified, the conditions are quite unfavorable to chemical reaction, and a fractional part only of the acid can be supposed to be effective. That some of it is effective will appear below, but we have no hesitation in saying, one, that under the actual conditions this amount will not injuriously affect the cement; and two, that *per contra* if the cement can react to neutralize all this free acid, the iron embedded in the concrete will be fully protected from corrosion.

Test Specimen.—To prove this by actual experiment, we made up very carefully a 2-in. cube of concrete, and three days after making it, a drip of distilled water was started over it, which was maintained

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for one week. An analysis of the solids dissolved out in this way showed that the proportions of lime and sulphuric acid were:

"CaO	13.28 per cent.
SO_3	13.80 " "

"The sulphuric acid present can combine with but 9.66 parts of lime, leaving 3.62 parts in excess of that required to neutralize the acid. There was no acid reaction, and the analysis shows that under favorable conditions there will be none.

"*Practical Test.*—For a practical test of steel embedded in a 1-to-2-to-5 concrete as used in New York, we made up from the samples furnished a concrete proportioned as follows: 540 grammes of cement; 1 080 grammes of sand; 2 700 grammes of cinder.

"*Effect of Concrete on Naked Plate.*—This we packed in a box 6 by 8 by 5 in. in dimensions. In the center of this concrete, we placed a piece of thin and highly polished steel plate obtained by cutting and flattening out a piece of seamless bicycle tubing. After 28 days, the concrete was broken up and the plate removed. This plate we send you herewith. You will observe that the condition of its two sides are in marked contrast. On the bottom, the mortar was in contact with nearly the entire surface of the plate, and wherever it was thus in contact the metal is as bright as silver. On the top, where the mortar did not protect it, except in small spots, the surface of the plate is rusted and pitted by corrosion. This difference in condition of the two sides is due to the well-known fact that in tamping concrete the stone settles to the bottom and the mortar rises to the top. This is especially marked in meager concrete, in which the mortar is insufficient to fill the voids in the stones, which is the case in this mixture. In tamping the plate in position, the mortar has risen to cover its bottom, while in tamping the concrete from the top of the box, the cinder has settled down on top of the plate to the exclusion of the mortar.

"The top of this plate undoubtedly shows the corrosive effects of sulphuric acid and exhibits corrosive effects which, in a damp atmosphere, would be continuous to its complete ultimate destruction. The reason for this is that sulphuric acid or sulphates which will react with iron, act, in the presence of moisture, as continuous conveyors of oxygen to the metal. The acid first reacts upon the iron to form sulphate, and this, by a second reaction, becomes oxide, releasing the acid to again attack other particles of metal. In this way a small amount of free acid may, under favorable conditions, rapidly pit and destroy unprotected ironwork. This effect is exhibited in the rapid destruction of cast-iron pipes laid in cinder banks. The percentage of acid in the cinder, as the analyses given will show, is quite small, and the amount which is capable of chemical reaction, much smaller yet. But it is sufficient, by successive oxidation, to destroy the pipes rapidly. The same effect is seen in the rapid corrosion of ironwork from the sulphur fumes in smoke.

"*Conclusions.*—The inevitable conclusions to be drawn from these considerations are, one, that unprotected ironwork may be rapidly corroded by contact with this cinder in a damp atmosphere; but, two, that the iron will be completely protected from such attack if the cinders are embedded in a sufficient matrix of cement mortar.

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"A 1-to-2-to-5 concrete, as now used, does not afford a sufficient matrix for the cinder, and unprotected iron in this concrete will corrode, and, in the presence of moisture, may corrode rapidly.

"Theoretical Proportion for a Full Concrete.—We suggest, then, that an amelioration of the concrete is desirable, increasing the relative proportions of the mortar. Thus, we have determined by experiment that the voids in the cinder are 54.5% of its bulk. These voids should be completely filled with mortar, allowing some excess of mortar besides, because its distribution can never be perfectly effected. The voids in the sand are 35% of its volume. Assuming, then, that the cement is to fill the voids in the sand and the sand the voids in the stone, the proportions of aggregates for a 'full' concrete of these materials would be, theoretically, 1 to 2.85 to 5.20. Allowing a little excess of mortar for practical work, as is necessary, this would give, for a working formula for full concrete, 1 cement, 2.75 sand, 4.75 cinder.

"Formula Suggested.—This formula we suggest for adoption in using this cinder concrete in your fire-proof-floor-arch system.

"This concrete should protect the iron from corrosion by either sulphuric or carbonic acid. The latter, it will be remembered, we dismissed at the outset of the discussion as present and able to act if conditions favored. In the presence of an impermeable concrete, as has been recommended, containing a considerable percentage of calcium hydrate, no rusting from carbonic acid and water is to be expected in your floor arch system. If it is impracticable to use such a mixture as has been suggested, under the building laws of the City of New York, we recommend, then, that a mortar of 1 cement to 2.75 sand be spread first in contact with the wire mesh of your arch system and with the surfaces of the beams. The cinder concrete, in proportion of 1 to 2 to 5, can then follow in usual course.

"We trust the above will fully cover the ground you desired us to investigate, and that this report will be satisfactory to you.

"Very respectfully submitted,

"BOOTH, GARRETT & BLAIR."

"MESSRS. BOOTH, GARRETT & BLAIR,
"404-406 Locust Street, Philadelphia, Pa.

"GENTLEMEN.—We hereby acknowledge receipt of your special report on the effects produced by cinder concrete on iron embedded in it; also a piece of sheet steel tested. We are much pleased with the report, and we consider the amount of sulphuric acid and sulphides found in the cinder phenomenally small. We are a little surprised at the practical results which you have obtained in the sample of sheet steel sent us. It is evident that where the material is porous and the interstices not filled in solid with cement mortar, that the moisture in the concrete produces oxidation, while, on the other hand, where the cement mortar covers the entire surface of the iron, it serves as a very effective protecting material and prevents oxidation. These results complicate our problem considerably.

"If we should adopt a concrete such as you have suggested, using proportions of 1 of cement to 2.75 of sand and 4.75 of cinder, thereby

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securing a material which would have the interstices filled in with cementing material and be a solid, compact mass, we would have a concrete that would not resist violent changes in temperature. It has been shown very conclusively in fire tests that when a solid material like stone concrete, well rammed, so that the interstices are all filled with cement mortar, is heated to a glowing red heat and then suddenly cooled by the application of the fire stream, the material is unable to stand such a violent change in temperature and invariably cracks and disintegrates. The same is true as regards material of this character when subjected to freezing. It would also be necessary in our system of fire-proof flooring to use a firm and water-tight centering, which would not only prevent the water from dripping out, but which would also make practicable considerable ramming of the material. The moisture would thus be retained in the concrete, and, if subjected to freezing weather, would freeze and crack and might be very seriously injured.

"One of the claims which we now make is that our material, as we manipulate it and apply it, is porous and will sustain successfully the most violent changes of temperature without cracking. Now, according to your report, if we would use a material which will successfully protect the iron and prevent corrosion, we must modify it so that it will not give the best fire- and frost-resisting properties, besides making it very much heavier. In making a change of this kind, we would be losing many practical and economical advantages. On the other hand, if we continue to use the material as we do, there is the possibility of an initial oxidation, which, as we understand it, will last until the moisture disappears. It would, therefore, be interesting to know:

"1st. Whether or not this oxidation would continue in the absence of moisture, which condition would occur when the floors are thoroughly and completely dried and the building is under roof.

"2nd. Whether an initial coat of paint, such as is commercially applied to all ironwork, would not prevent oxidation until the moisture contained in the concrete disappears.

"We would be pleased to have you write us and express an opinion as to the two points above referred to, as well as any further suggestions that might occur to you.

"Very truly yours,

"JOHN A. ROEBLING'S SONS CO."

"Dictated by A. L. A. H."

"Philadelphia, March 11, 1898.

"MESSRS. JOHN A. ROEBLING'S SONS CO.,

"117-119 Liberty Street, New York.

"GENTLEMEN.—We are duly in receipt of your favor of the 5th, which has had our careful attention. We think the points which you have raised are very well taken, and fully appreciate their practical bearings.

"We have carefully considered the questions you raise, to wit:

"1st. Whether or not oxidation would continue in the absence of moisture, which condition would occur when the floors are thoroughly and completely dried and the building is under roof.

"2nd. Whether an initial coat of paint, such as is commercially applied to all ironwork, would not prevent oxidation until the moisture contained in the concrete disappears. Mr. Himmelwright.

"And in replying to them in order, would say, first, evidently any corrosion of metal must go on very slowly in the dry atmosphere between the floor and the ceiling of a building. The pitting shown on the plate is, undoubtedly, due to acid attack, and this would probably slowly progress under these conditions; the rusting which comes from the water probably not; second, we are disposed to think, after carefully weighing the facts, that the iron would be protected by a good paint covering. In other words, we should hold that you could take this position, to wit, that if the manufacturer of the iron protected his material by a first-class quality of paint, you would guarantee that no corrosion would take place. We have under way a few simple experiments to cover these points and shall advise you of the results.

"We take it for granted that the painting of the iron work would not be within your province, and that therefore you could not decide what the paint should be. For your wire mesh and ribs, however, we would suggest that a paint of the asphaltum variety, free from oils, which can be dried up by acids or alkalies, would probably be better than an oil paint.

"Yours respectfully,

"BOOTH, GARRETT & BLAIR."

"Philadelphia, March 28, 1898.

"MESSRS. JOHN A. ROEBLING'S SONS CO.,

"117 Liberty Street, New York City.

"GENTLEMEN.—We beg to summarize the various reports heretofore made to you covering the question submitted to us for examination, to wit: 'Is any injurious effect to be expected from the use of cinder concrete in contact with steel beams in fire-proof floor arches?'

"We find:

"(A) 1. The sulphuric acid in the cinder will average 0.20 per cent. (two-tenths of one per cent.)

"2. The sulphur as sulphides in the cinder will average 0.20 per cent. (two-tenths of one per cent.)

"And that,

"(B) 1. These percentages of sulphuric acid and sulphur will not injuriously affect the cement.

"2. They will not injuriously affect good paint.

"3. They are sufficient to corrode unprotected ironwork more or less rapidly, depending on the presence or absence of moisture.

"We also find:

"(C) 1. That the mortar of the concrete, wherever it is in contact with the iron, fully protects it from oxidation.

"2. But that the proportions of 1 to 2 to 5 do not make a full concrete, and, hence, would not fully protect bare iron surfaces.

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"It follows from this:

"(D) 1. That there would be some oxidation of bare iron or steel beams in this concrete.

"2. That in the ordinary dry atmosphere between floors of a fire-proof building this oxidation could occur over a limited area, and would be extremely slow.

"3. That bare iron would be fully protected by the use of a 'full' concrete (say 1 to 2.75 to 4.75).

"4. That the iron is fully protected by good paint (red lead and linseed oil for example).

"The conclusion we reach is that no injurious effects are to be anticipated from the use of this concrete above ground in the ordinary practice of construction now followed for steel frame buildings.

"Yours respectfully,

"BOOTH, GARRETT & BLAIR."

From this report, it will be seen that the preliminary investigation stopped practically at the point where Professor Norton's investigation ended. At the suggestion of the writer, a further investigation was made to determine additional and equally important facts. The correctness of the final conclusions of Messrs. Booth, Garrett and Blair have been fully borne out by the experience of the last 15 years.

It is apparent from the final conclusions that a porous cinder concrete will not cause the corrosion of structural steel, and a porous concrete, therefore, is a perfectly safe and economical material to use for fire-proof floors. This fact has been verified repeatedly in the razing of buildings, and in cutting openings in floors in numerous buildings where segmental arches of porous cinder concrete were used for the floor construction.

It is equally true, and experience has also confirmed this conclusion, that there is an initial oxidation in naked steel embedded in the concrete adjacent to all voids; but this oxidation progresses so slowly in the absence of moisture that it is practically negligible. Consequently, even in reinforced slab construction, a full or dense concrete, with its greater weight, though desirable, is not absolutely essential as long as the required bond stress and strength are obtained.

Another interesting field for investigation, with large promise of greater fire resistance without material loss in strength, is experimentation with reinforcing bars of such section as will obtain greater protection from the surrounding concrete. The writer refers to flat bars, wire trussing, strips of expanded metal, etc., set on edge and extending 2 in. or more into the concrete from the under side.

The writer made a number of fire and water tests with flat bars, 2 by $\frac{1}{8}$ in. and 2 by $\frac{3}{16}$ in., set on edge, with the under side of the bar $\frac{1}{2}$ in. above the under side of the slab. Very interesting and remarkable results were obtained with this type of reinforcement. The fire

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resistance of the construction was enormously increased and, strange to say, the strength, as determined by numerous tests, was approximately the same as that of the conventional section of bar, round or square, placed in the usual position of $\frac{3}{4}$ in. from the under side of the concrete slab. This result could only be accounted for by the fact that a large surface for bond stress was presented by a bar of this shape, and the additional fact that a flat bar set on edge had considerable value as a beam.

As a result of the writer's studies and investigations with this type of bar, it was adopted by the Roebling Construction Company, and, as the Roebling System "B", was installed in nearly 1 000 public buildings, offices, and hospitals throughout the United States during 1903 to 1908. The superior fire resistance of this method was thoroughly demonstrated in the San Francisco fire in the following manner:

There were perhaps 20 buildings occupied as offices and apartment houses, in which flat, reinforced-slab floor construction was used. Four or five of these buildings were fire-proofed by the Roebling System "B", and the rest were flat-slab floor construction with different methods of reinforcing, including meshes and rods of different varieties, but all placed near the under surface of the slab. In this class of buildings, the fire tests, as represented by the combustion of inflammable floor finish, trim, furniture, and furnishings, were only approximately $\frac{1}{2}$ to $\frac{3}{4}$ hour in duration. Temperatures from 1 800 to 2 200° Fahr. were obtained. This fire test, without the application of water, was sufficient to cause a considerable percentage of the floor arches in the ordinary methods of construction to sag under loads, such as safes, merchandise, etc., but such signs of weakness were not apparent in any of the buildings in which the "B" system of flooring, similarly loaded, was used. In some cases of other methods of flat-slab construction, as much as 30% of the entire floor area had sagged and had to be releveled or replaced when repairs were made.

The writer believes that it is possible to obtain largely increased fire resistance by these and similar methods of reinforcement, without any material sacrifice in strength or increase in cost, and that, in the near future, the advantage of superior fire resistance must receive the recognition which it deserves.

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FUNDAMENTAL PRINCIPLES OF PUBLIC UTILITY VALUATION

Discussion.*

By John W. Alvord, M. Am. Soc. C. E.

Mr. Alvord.

John W. Alvord,† M. Am. Soc. C. E. (by letter).—The writer is greatly indebted to the members of the Society who have discussed his paper. What little value there was in his own effort has been greatly enhanced, not only by the discussions which have agreed with his views, and the kindly approval generously accorded, but, as well, by members having differing view-points, who have taken the time and pains to register their honest dissent. All such contribution must cause the final truth to be more sharply thrown into relief.

This paper was written primarily to confine attention to the subject of valuation. Rate-making was avoided as far as possible, so as to limit attention to one subject. So much of the discussion, however, has inevitably drifted into the rate-making results of valuation that it may not be amiss to outline briefly some principles in utilizing valuation for rate-making purposes.

It is desirable to confine such statements to certain fundamentals as to which there is little or no room for controversy, and little or no controversy, in fact. This will tend to clear the ground for a consideration of the details presented by the discussion.

1. Rate-Making.—(*a*) This is a legislative function performed by legislatures, or by commissions or municipalities to which the legislatures have delegated administrative functions in connection with the exercise of this power. This is not a judicial function. Courts have not the power to make rates, and do not make rates.

* Author's closure of the paper by John W. Alvord, M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

† Chicago, Ill.

Mr. Alvord.

(*b*) The power to regulate rates is subject to the limitation that the rates shall be reasonable, that is, that they shall not exceed the fair value of the service to the consumer, and shall not be less than that which will afford a fair return on the value of the property.

(*c*) Reasonable rates are such as are just and fair, between the limits stated, after due consideration of all the pertinent circumstances.

The maximum of what the service is worth to the consumer and the minimum of what will afford a fair return to the utility constitute the limits within which the rate-making power may justly exercise "the flexible limit of judgment which belongs to the power to fix rates". (206 U. S., 26.) Different conclusions, both reasonable, may be reached by equally competent tribunals from the same facts. There is no mathematical rule.

"With that sort of evidence before them, rate experts of acknowledged ability and fairness, and each acting independently of the other, may not have reached identically the same conclusion. We do not know whether the results would have been approximately the same. For there is no possibility of solving the question as though it were a mathematical problem to which there could only be one correct answer." (222 U. S., 550.)

(*d*) Economic laws require value to be taken into account in performing the legislative function of rate-making. The money for investment in public utilities must be secured in a competitive market. It can only be induced to go into these utilities by the promise of a sufficiently attractive return. Practically every public utility constantly requires new money for extensions and additions. The question is not what return should a commission permit on money which is invested and cannot be withdrawn. It is rather, what return on the money already invested is required to attract the necessary new capital, if the public service is to remain unimpaired. Obviously, fundamental economic laws require, in the long run, that the return from these investments be equal to that which might be derived from other available investments, after adjusting differences of hazard and other variations in the surrounding circumstances.

2. Suits Assailing Rates.—The Constitution of the United States, as interpreted by the Courts, prohibits, as involving the taking of property without due process of law, rate schedules which will not afford a fair return.

(*a*) The limit of the jurisdiction of the Courts is to determine whether any specific rate schedule will afford such a return on the fair value of the property at the time in question, and if it will not, to set that schedule aside. The Courts have not the power to prescribe schedules of reasonable rates, or to establish schedules of reasonable rates to take the place of those which are set aside as unreasonable.

(*b*) In a proceeding before the Court, the fair value of the property used for the convenience of the public, or, to put it another way, the reasonable value of the property at the time it is being used for the public, is one of the ultimate facts which must be determined. As to this, it is said by the United States Supreme Court (230 U. S., 434): Mr. Alvord.

"The ascertainment of that value is not controlled by artificial rules. It is not a matter of formulas, but there must be a reasonable judgment, having its basis in a proper consideration of all relevant facts."

A proper appreciation of the radical distinction between the jurisdiction of the rate-making body on the one hand, and of the Courts on the other, will strongly tend to clarify the situation. Perhaps it may be said, without impropriety, that the jurisdiction of the regulating bodies ends where that of the Courts begins. The jurisdiction of the Courts is above the line of confiscation. The reasonable rate lies between the rate that is on the line of confiscation and the rate that is so high that the charge exceeds the value of the service. The jurisdiction of the Courts may be invoked when the regulating body has exceeded its jurisdiction, and has attempted to establish a rate that is confiscatory.

Within its jurisdiction, the regulating body has the right to take into account every factor which may legitimately affect the exercise of its discretion. The regulating body may, and should, take into account, not only the value at the time, but also the investment, what it would cost to reproduce the property new, the probable investments that will be required in the future, whether the service rendered by the plant is of 100% efficiency, the revenues which the plant has been earning and which it would probably earn under the proposed schedule of rates, and every other fact material to the determination of what, as between the utility on the one hand and the consumer on the other, would be reasonable and just.

Much of the confusion with reference to this matter has apparently grown out of the assumption that the regulating body has the power to take into account, in connection with the investment, only a single factor, which must be called value, whatever it is. Partly on this account, some who have believed that original cost should be the basis of rates, have advocated this as a measure of value; others, for the same reason, have advocated 100% of the reproduction new; others have advocated reproduction new less depreciation; and others have presented still different theories. As a matter of fact, every one of these factors may be material in connection with the establishment of a rate schedule, and may properly be considered under its own proper name, without an attempt to make it available through a forced definition of the word "value".

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On the other hand, when we go out of the domain of rate-regulation into the jurisdiction of the Courts, the rule is perfectly well established that the material fact is the value of the property, and the Courts have properly said that this cannot be determined by any formula. There is no formula which will show the value of a horse, or of a piece of real estate, or of any other property. In every case, there must be "reasonable judgment, having its basis in a proper consideration of all relevant facts". In announcing this rule, the Court has simply declared what is common experience. Value is, and should be, used in its ordinary, every-day sense. It is difficult to define it, but it is no more difficult than to define what is the value of a horse, or of a piece of real estate, although the determination of what is the value of the utility may present a much more difficult and complex question.

A careful consideration of these fundamental principles will clear away much of the confusion of mind observable in some of the discussion on this and the writer's prior paper, "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return";* in fact, this present paper should have preceded the former one, in order that both might be intelligible.

Turning now to some of the discussions of the present paper in detail, the writer has naturally been particularly interested in those which have taken issue with the paper, and though he regrets the necessity of the "last word" imposed on him by the closure, nevertheless, he would hope that some remark, particularly on some of the important dissenting discussions, will not be without further helpfulness in the matter.

Mr. Haupt's review of the Monongahela Navigation Company's case is of great interest, for it is to be remembered that this is one of the leading cases in the Supreme Court on franchise value. (Monongahela Navigation Co. *vs.* U. S., 148 U. S., 312.)

Mr. Jewel, Mr. Dana, and Mr. Lavis, though cordially assenting to the writer's views, have each and all made valuable criticisms and suggestions which are appreciated.

The discussion by Mr. Mayer is interesting and valuable. He has come to the subject from the standpoint of the theoretical and logical economist, and the writer has read what he has to say with interest and profit. A second reading, however, has raised some question as to the soundness of some of his propositions, and caused some regret that he is not as helpfully constructive in his economic ideas as might be desired. For instance, he criticises all the definitions of value offered, and it must be admitted that most of his criticisms have force, but he does not offer any definition of value to take the place of those he apparently destroys, and one might be puzzled to know, after a careful

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

reading, what his brief definition of value would be. The writer concludes, after reviewing Mr. Mayer's closing discussion* on his paper, "The Just Value of Monopolies, and the Regulation of the Prices of Their Products", that it may be something like this: Mr. Alvord.

"Value is a measure of the supply required for a human need."

Perhaps this is a composite of what is in the minds of Mr. Mayer and the writer, or perhaps it is wholly a pre-occupation of the writer's. Be this as it may, it is offered for what it is worth.

The writer has been unable to grasp Mr. Mayer's criticism on his own adopted definition of value (page 3203†), that it is contradictory, although he has read over and thought about the criticism not a little.

Mr. Mayer thinks that an important factor has been omitted, in considering values, by neglecting to mention the importance of the character of the future regulation. It would undoubtedly add much, not only to our ability to value correctly, but to our ability to form proper judgments in all other aspects of life, were we certainly apprised of that interesting class of facts which lie wholly in the future; but what the character of future regulation will be is a factor which no one can ever certainly know, and, therefore, it must be a speculation in any case. It is this fact, more than perhaps any other, that causes value to be a matter of opinion. All value is predicated fundamentally on what the future will bring forth, as Mr. Mayer suggests, and perhaps it would have been well if the paper had dwelt on that fact to a greater extent, but the paper is not a thesis on theoretical economics, it is an attempt to point out the practical governing conditions under which we must proceed to accomplish something specific in the light of our restricted knowledge, and, for that reason, we must build on what we have.

Other suggestions of Mr. Mayer might be commented on as follows. He very properly says:

(*a*) That the foundation of valuation rests on equity and justice. (Page 3203.†) This is quite true, but the formulated equity and justice under which we must actually work are the Constitution, the law, and the interpretation by the Courts, which are serious and able attempts to make equity and justice practically workable, and we do not want to take time now to consider what would happen if we did not have these valuable precedents and rules of practice in our work.

(*b*) Future legislation does not concern us, because, as Mr. Mayer says, we cannot know what it is to be. (Page 3208.†) We must assume from the past, therefore, that it will be reasonable and fair, so as to attract capital into the utility field.

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* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 492.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

Mr. Alvord.

(*c*) It is not especially valuable to us to know that every utility, even of the same kind, should have a theoretical fair return, different from any other utility, and predicated on what like competitive business would produce. This may be interesting theoretically, but all we can hope to accomplish practically is some approach to justice between the buyer and seller of specific service, which can be approximated by comparatively simple and reasonable methods in each instance. (Page 3213.*)

(*d*) In holding that it would be absurd to attempt to find the depreciated value of an obsolete plant by the reproduction process, Mr. Mayer goes to an extreme case to prove his point. (Page 3210.*) It is unusual to find a wholly obsolete plant in service. Normally, plants are only partly obsolete, and usually only slightly so, and it is certainly of great advantage to go through the reproduction process in such cases, as it affords much valuable insight into structural value to examine a plant in detail.

(*e*) Mr. Mayer says: The true worth of any utility is the present worth of its future net earnings. (Page 3209.*) Presumably this is sound, if the physical plant is amortized at the end of the earning period, but of what interest is it to us practically; we can never hope to know by any possibility the present worth of the future net earnings. What looms large to us as a sheet-anchor, in our attempt to lay hold of something practically helpful, is that most utilities have the largest portion of their investment in the physical plant, and it is approximately possible to tell what this plant is worth, judging by the need for its output, and it is also possible to tell what it will cost to finance it on a paying basis, also, in some cases, what it has cost. With these facts and other surrounding circumstances, it is further possible to answer approximately the question, what is its present fair value, always supposing, of course, that it is not to be destroyed by unjust legislation, wrecked by careless management, or engulfed by an earthquake.

Mr. Mayer has said many valuable things, and has pointed out particularly well, in the writer's opinion:

(1) The fallacy of public regulation of rates, without incentive to good management;
(2) The importance of comparing "fair return" with that obtained in kindred enterprises; and
(3) The fact that the present tendencies of regulation are inevitably leading to guaranteeing the investment and operating the property by the commissions.

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* *Proceedings*, Am. Soc. C. E., for December, 1914.

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Mr. Burgess has presented a very effective plea for more deference to evidence of past cost, and the writer would cheerfully acknowledge that, under most normal conditions, much of what he says is reasonable, and under certain exceptional conditions, even more could be said in behalf of past cost.

It is very difficult, in such a discussion as this, to select the proper perspective for the consideration of two cost methods, each of which will vary in preponderance in different cases. Past-cost evidence has undoubtedly been over-emphasized, by some of our commissions, where it should not have been, and, in an attempt to offset this tendency, the paper may not have adequately conveyed the impression which the writer really holds, that past cost always has some remote usefulness in valuation work, and has an undoubted value where it is not too remote from present conditions. The nearer past cost is to the present time, the more and more valuable it becomes, until, finally, in considering a plant which has just been built, past cost, of course, must have a large preponderating influence in the summing up. The writer had occasion recently to value two water-works plants; one had been built within 8 years, the other was nearly 100 years old. The past cost was known in both cases, but the older plant, originally built with bored logs, had no trace of its original construction left. Obviously, past cost, under these conditions, did not throw much light on present value; but, in the other case, past cost was so recent and so complete, that reproduction cost new would have been little different in result.

Mr. Burgess has made it clear that, in very old plants, past cost should not have too much weight, and it is believed that there is no essential difference between his views as stated, and those embodied in the paper, except that the perspective in the paper on this point is perhaps not as fully and well brought out as it might have been.

Mr. Newell, in his brief discussion, makes the following astonishing statement: "Under regulation, the duty of the public service commission is to determine, not what the value of the property is, but what it ought to be."

The writer cannot help reflecting what a beautiful world this would be if we, as self-appointed commissions, could determine the value of our individual property, and fix it, not on the basis of what it is, but what it ought to be.

Mr. Harte, as well as Mr. Mayer and Mr. Dana, has criticized very intelligently the definitions which the writer quoted from legal authorities on valuation. It is realized that these definitions are not as satisfactory as could be wished. Possibly "value" is not susceptible of a clear, brief definition, but it would seem to be desirable that one should be determined, if possible. Thus far, the writer has been less satisfied with those definitions which are found in books on economics than with those which he has submitted from legal opinions.

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In the writer's mind, the difficulty which disturbs such men as Mr. Humphreys, Mr. Grunsky, and Mr. Knox (pages 137 and 245*), who, in brief, believe depreciation should not be deducted from cost new, and that rates should be adjusted to 100% value, is this: That they have approached the problem of valuation from the view-point that the value of the property is of controlling importance in fixing rates, and that the commissions must and should protect the original investment, whereas the view-point of the law is that it is the value of the property now that is considered only as a lower limit beyond which there must not be confiscation.

They neglect to observe that the value of the property now is useful chiefly in defining this lower limit of fair return, as distinguished from confiscation, and that the field, or fair rates, lies between the line of confiscatory return, on the one hand, and the worth of the service to the consumer, as an upper limit, on the other.

If the State were to undertake, by suitable rates only, to protect the original investment intact, without allowing it to grow by so-called "unearned increments", or to lessen by such losses as may occur, then it is obvious, as these gentlemen contend, that depreciation should not be deducted from the original cost for rate-making purposes.

The writer, however, has attempted to show in the present paper, that no such condition of protection by the State of an original investment is contemplated by the law under which we are working. What is expected under the rule now in use is that the original investor shall accept all the inevitable ups and downs to which his original investment may be subjected, and that the rates, where the rate-making powers are exercised, shall lie within the field limited, on the one hand, by the worth of the service to the consumer, and, on the other, by the necessity of avoiding confiscatory return on the fair value of the property at the time such powers are invoked. At such time, this fair value must, therefore, under this theory, include all the appreciations as well as exclude all the depreciations to which the property has been subjected since the original investment was made.

Mr. Humphreys argues (page 141*) because renewals are numerous and frequent in some kinds of property, and, owing to short life and small individual amount, such depreciations are, for convenience, converted into operating expenses, that the property as a whole ought to be considered as having original value without depreciation deduction. This may be a practical way to handle some forms of depreciation in certain extreme cases, but it does not alter the general principle that losses of value which have occurred since the original investment was made should be considered, together with increases in those values, in determining the value of the property at any given date.

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* *Proceedings*, Am. Soc. C. E., for January, 1915.

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For example, if a prosperous and growing main-line railroad has a short but expensive branch to a mine which becomes worked out and ceases to operate, and this branch, in consequence, has become of no further use, present or prospective, we speak of it as having depreciated in value, and we must take that fact into consideration, as well as the increased value of the main line, in arriving at the value of the railroad property as a whole as of the date of valuation.

If, in the foregoing case, by agreement with the State, we were considering only the protection of the original investment, we should have to continue to include the branch at its original investment cost, and deduct from the main line all the increased value it had acquired by its prosperous growth.

The unwisdom of considering the State as a guarantor of investment, through its rate-regulating powers, is too obvious to be discussed at length. The State cannot engage in a policy of protecting investment, even by the roundabout method of establishing fair return on original cost, as a settled policy. To do this would be to take all incentive for economy or prudence from the private investor, and to assume all the uncertainties and vicissitudes of an unknown future in the business world. Hence the rule has been adopted by the Courts, in interpreting the Constitution and the law, that it is the fair value of the property as of the date when such rates must be determined, and it requires little reflection to see that any other course would not be sound or wise public policy conducive to justice and the public welfare.

Mr. Knox continues, in his discussion, his dissent to the writer's former paper* on depreciation, although, in the light of the closure to that paper, and the present paper, the modification to his thought is apparent.

Among other matters, in the course of his argument, Mr. Knox strives to create what seems to the writer to be a new definition of depreciation, namely, that it is limited to physical wear and tear only. This idea seems to prepossess a number of engineers writing on this subject, but is certainly without warrant. Webster gives no such restricted definition. Common use does not warrant such a limitation. When we speak of a house depreciating, we do not consider that it is lessened in worth only by physical wear and tear. Age, changing styles, lessened demand, and other causes have usually much more to do with its loss of value than wear and tear.

Practical work in appraisals very quickly convinces one that lessened value arises from many causes other than wear or physical deterioration, and these other causes have usually much greater influence in depreciating property. Take, for instance, the illustration previously used, of the railway having a branch to a worked-out mine.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

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We could assume that the branch line, not anticipating abandonment, had recently been renewed as to its rails, ties, and bridges, so that it was physically as good as new. This would not have prevented its large loss of value as a branch, due to lack of need for its facilities. The depreciation might have been a little greater if the ties and rails were old, because the scrap value would have been less, but the main factor of loss is to be found in changed conditions of general need for the service, and all the arguments that Mr. Knox has suggested do not alter the condition that if we must value such a property, and value it as of the present time, we must deduct, as a depreciation, the loss of value of the branch line in considering the fair present value of the railroad as a whole.

Mr. Knox (page 147*) can find no instance in which the Courts have decided that depreciation reserves are to be computed on the sinking-fund basis, or that "physical" depreciation must be deducted from value new in rate-making appraisals.

If Mr. Knox expects to find such explicit directions as to engineering and economic details of valuation, such as the use of sinking funds, from the Courts, he will be disappointed. The Courts do not usually presume to go so far outside of their proper domain. Engineers sometimes unconsciously presume to instruct the Courts as to law, but few Courts presume to instruct engineers as to detailed engineering methods. The Courts interpret the law and pass on the evidence, but it is for the engineer and the economist to formulate rational methods of presenting the evidence.

As to how loss of value in structures and property is to be determined, is, of course, an interesting question, on which opinions can honestly differ. The assumption that depreciation as a loss in value is often well measured by the accumulation of an amortization fund, is a method of basing such estimating which might be properly debated and discussed. Obviously, it is not true in every instance, or in every kind of a utility, or in any single structure at every point in its career; but, as an attempt to average a great variety of conditions, it has hitherto been accepted by a large majority of appraisers as fairly well answering general purposes as a beginning from which to reason. To the writer's knowledge, no one hitherto has accepted it as a hard and fast rule or principle. Possibly, this tendency to accept it as a mathematical solution has also troubled some of those who study depreciation discussions.

The question of deducting depreciation from cost new, however, is fundamental to a proper value, and, as a matter of fact, for the past 15 years, engineers and attorneys have been arguing for its deduction, and Courts have been approving the practice of subtracting the depre-

* *Proceedings*, Am. Soc. C. E., for January, 1915.

ciation from reproduction cost new in every case brought before them, without a single exception which the writer can recall. Engineering appraisal boards, so far as the writer is aware, have pursued the same practices. In water-works appraisal, the question was discussed and settled more than 15 years ago, and has never been seriously controverted since, until Mr. Grunsky's paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates",* appeared, in 1912, before this Society. That paper ignored the settled practice, so carefully thought out and already well argued and passed on, and it seemed to the writer that the original reasoning by which the question had been settled should be stated for record. This he did in a former paper† before the Society. Mr. Grunsky argues in his discussion (page 151‡) that as net profits fix the value of a property, the value should not be used in fixing rates, for this is reasoning in a circle. From this, he apparently argues that the whole present method of fixing value is wrong, and Court decisions based on it are erroneous and should be changed. Mr. Alvord.

Mr. Grunsky must realize that, in taking such a position, he is passing adversely on a proposition which has been settled by a large number of trained minds—official, judicial, and expert—who have considered this problem carefully for many years.

The writer has endeavored to point out in his paper (page 2406§) that we do not reason in a circle by fixing value first, because we have, in all ordinary types of utilities, a physical plant as a basis, and this, together with a fair return (which is assumed to be sufficient to cause capital to flow freely into the business), will fix the fair value of the property, and fix it so effectively that there can be only one value for a given physical plant and a given minimum fair return. This fundamental proposition was the subject of much discussion among appraisers some years ago, but was found to be capable of simple solution, and has in no wise troubled Courts or appraisers since. Hence, we need not overturn anything—not even so well established a practice as that of the Supreme Court—in the effort to establish justice.

Mr. Kiersted frankly dissents from some of the principles which the paper has endeavored to point out as fundamental. Briefly summarized as to his important points, he would seem to hold:

(*a*) That equity and justice are the foundations of valuation work.

(*b*) That the reproduction theory really embraces three distinct methods of valuation:

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* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

† "The Depreciation of Public Utility Properties as Affecting Their Valuation, and Fair Return", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

‡ *Proceedings*, Am. Soc. C. E., for January, 1915.

§ *Proceedings*, Am. Soc. C. E., for October, 1914.

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(1).—Present-day conditions and prices;
(2).—Original conditions and present-day prices; and
(3).—Original conditions and original prices.

(*c*) That there may be no practical distinction between cost and value in monopolistic utilities.

(*d*) That the determination of value for purchase and sale may be different from that for rate-making.

For the most part, the paper itself is an attempt to answer these positions. Mr. Kiersted's view-point being obviously that it is an investment which desirably must be protected and assured, apparently he does not see the usefulness of the law's attitude that the fair value of the property as of to-day should be the governing factor.

Mr. Burns' discussion contains a very good illustration of fluctuating value, which seems to answer Mr. Kiersted's contention that cost and value may be practically identical. Something, however, may be said briefly as to the other points raised:

(*a*) Equity and justice are most assuredly the foundation of appraisal work, but everybody's idea of equity and justice is, unfortunately, not the same, consequently, the American people have formulated their fundamental ideas of equity and justice into a Constitution and laws, which, as interpreted by the Courts, are guides in cases of differing opinion. It is obvious that we must work under these formulas until they are changed by general consent.

(*b*) In outlining three methods by which reproduction processes may be carried out, Mr. Kiersted uses the term "reproduction" in an apparently different sense than it is ordinarily used. He says (page 417*): "The reproduction theory really embraces three distinct methods of valuation," which he goes on to describe.

The writer's definition of reproduction (page 2392†) is: "An estimate of the cost of re-creating a property at the present time under conditions that are humanly possible and practicable."

Mr. Kiersted's conception, as nearly as can be defined from an examination, especially of the second and third methods which he outlines for its use, would seem to be: The re-creation of the whole or a part of the whole of a given property, without reference to physical possibilities or human limitations.

In the first case which he outlines—that of re-creating a property under present-day conditions and prices—he fairly well describes reproduction as appraisers generally understand it, but, in the second method cited, he outlines a form of re-creation such as is not, humanly speaking, possible, namely, the re-creation under original conditions, but at present-day prices. Mr. Kiersted has had a long and successful

* *Proceedings*, Am. Soc. C. E., for February, 1915.
† *Proceedings*, Am. Soc. C. E., for October, 1914.

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experience in building and remodeling water-works properties, but he certainly would not claim to be capable of building plants at prices to-day under conditions which existed 20 or 30 years ago.

An engineer who could construct the New York Central Terminal, in New York City, at present-day prices, but under conditions of population and land values which existed when the old New York and Harlem Railroad was first projected, would promptly have his hands full of business from thousands of clients.

Such a so-called reproduction proposition does not aid in finding the value to-day of the New York Central Terminal. It merely confuses the question. What we want to know is either what that terminal has actually cost in investment, with its several re-buildings, all told, in the long years it has been developing, or what it would cost to reproduce it to-day under present conditions. Nothing of a humanly impossible character halfway between these two rational inquiries throws any real light on what it is worth, or helps us in the least to base our ideas of value on workable facts and sound fundamental conditions.

Nor is Mr. Kiersted's third method of so-called reproduction entirely free from the same kind of objection. On the face of it, it is the retracing of the past investment, or, in other words, the attempt to substitute for actual book cost an estimate of book cost which will be complete and presumably as good; but, as Mr. Kiersted defines it, it is not even the reproduction of the past cost, because, as he says, it would take account only of the property existing at the present time, useful for utility purposes, and thus exclude all obsolete structures or other investment originally necessary, but now non-existent.

The same criticism, therefore, would hold here, that lies against the second so-called reproduction method. It is not original conditions; it is not present conditions; it is (to use Mr. Kiersted's own words) an unreliable phantom that lies between the two and can never be an actuality. It gives us nothing which is humanly possible of accomplishment; nothing for a foundation of reality on which intelligently to build an opinion as to value, and, therefore, it is misleading in so far as it purports to be evidence.

Unfortunately, it is true, as Mr. Kiersted says, that, in the past, many valuations have been a conglomerate of the three methods, but this is a regrettable stage of progress from which we must obviously emerge as promptly as possible into a domain of more rational thought and procedure. Appraisal work in the future must advance to the point where the engineer can give a sound reason for every position that he holds and for every step that he takes, or it will continue to be as unscientific in the future as it has unfortunately been in many cases in the past.

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One further point in Mr. Kiersted's discussion remains to be noted, that is, his obvious belief that value for rate-making may be different from business value. This apparently arises from his strong conviction that in appraising a property he is protecting an investment rather than finding its value as of to-day.

If the State, through its utility commission, were proposing to an investor the protection of his original investment by means of fair rates on its actual original cost, then it might easily happen, after a lapse of years, that the artificial value legislated into the property by the rate-making power might create a value for the property greater than its real value without legislative stimulus. This would not mean that the property had two values at the same time, but that it might have widely different values at two different times, dependent on the legislation of the State. The State, however, does not make any such proposition, and it has been suggested, and is very easily seen, that it would not be a safe or wise policy. Imagine a law being passed regulating rents for business blocks in a city to a fair return on the original investment, and the State and the investors contracting with each other to the end that, in view of regulated rents, the State would accept the original investments as a safe continuing basis for fair return!

Suppose that in one-half of the city there should be a great shrinkage of values due to the gradual removal of the population to the second half, while, in the second half, there should be a marked increase in prosperity following the changing conditions; what would happen? The property in that part of the city of lessening value—the first half—would be depopulated faster than ever, and the State would be unable to fulfill its part of the agreement, but in the second half—or prosperous part of town—great injustice would have been done to the owner in depriving him of his enhanced values.

In a similar manner, a State cannot assume the policy of fixing rates on a large number of public utilities within its borders on the theory that it will protect their original investment through adequate returns. If it does, it will do itself an injustice in the case of those properties having diminishing values, as well as do injustice to owners of those utilities which may come to have increasing values.

As has been said before, it is for these reasons, as well as others of equal importance, that, in the regulation of utilities, the burden or reward, or loss or gain in value of a property through a term of years, must, as a matter of sound policy, rest on the private owner. Hence the rule that the value of the property as of to-day is to be the governing factor.

If the value of the property as of to-day is the governing factor in fixing rates, there can be only one value for rate-making, or for sale to the city, or business purposes, and the test of this truth lies

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in the fact that all such properties live and grow by reason of the new capital that has already flowed and should continue to flow into them; and, in the long run, the fair return must be fixed with the end in view that capital will be attracted to the utility with the expectation that it will earn a return adequate to the obligations and hazards involved, and, at the same time, have opportunity of reasonable increase in value. When this is the case (and it must normally be the case, or utilities would not exist), the value for rate-making is also the value for either sale or purchase; for, were it not the case, such property would in time sicken and die if just rates were not replaced.

To confirm this truth further, let it be imagined, if possible, that a utility property had two values at the same time, one for rate-making purposes and one for sale purposes. Who would pay the value for sale purposes if it was materially higher than that on which the rates would yield a fair return; or who would willingly sell a plant if the sale value was arbitrarily fixed lower than the rates indicated was profitable? Evidently, value can be legislated temporarily into or out of a public utility property to some extent, but two values for the same property under the same conditions is apparently unthinkable.

In conclusion, the writer hopes that those who have so kindly discussed his paper will take in good part his earnestness in defending the point of view that we must not neglect to study the legal aspect of valuation if we would do constructive work in valuation matters. It is not only the writer's view, but is that of all the ablest engineers, economists, and lawyers with whom he has been associated, and he will willingly confess that for many years valuation as an art had no co-ordination for him as a purely engineering problem. It was not until he began to get an inkling of the fundamental ideas of justice, as embodied in the law and its theory, that he began to perceive that valuation as an art had underlying principles which gave it unity and intelligent purpose.

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT

Discussion.*

By Messrs. C. E. Grunsky and Carl J. Rhodin.

Mr. Grunsky.

C. E. Grunsky,† M. Am. Soc. C. E. (by letter).—The writer is sorry that what he said in his previous discussion should have been construed as a reflection either on the course taken by the City Engineer of San Francisco in calling for a board of consulting engineers, or by these engineers who are defending themselves against a fancied imputation of being political engineers. The writer has been on the best of terms with each of these engineers; they have his unqualified respect and confidence; and he would not intentionally have said anything to their discredit. His only purpose in referring to the appointment of consulting engineers was to show that no intention was apparent on the part of the present administration to create a Water Board, and that the present plan of managing municipally owned utilities would probably be continued.

The statements from the writer's earlier discussion, which are objected to, and which have been quoted, should be read in connection with the following which was a part thereof:

"Nevertheless, the danger should be recognized that competent management of to-day under the political system may be followed by incompetent or even corrupt management to-morrow."

Neither in this statement, nor in a preceding use of the term "political", is there any intent to cast an unfavorable reflection on

* Discussion of the paper by H. T. Cory, M. Am. Soc. C. E., continued from April, 1915, *Proceedings*.

† San Francisco, Cal. Previous discussion of this paper by Mr. Grunsky was printed in December, 1914, *Proceedings*.

Mr. Grunsky.

those who happen to occupy official positions. The instability of their tenure of office, however, is self-evident, and the large number of departments or sub-departments of the City's Government which have been and are assisting in directing the acquisition of a municipal water supply, and will have a voice in its management, condemn the system. No evidence is yet apparent that the creation of a stable Water Board is under contemplation, and it was to show this fact, and for this purpose only, that reference was made to the appointment of a board of engineers by the City at the request of the City Engineer. The writer agrees with those who state that it was better to have the consulting engineers selected by the City Engineer, who understands the problems to be passed on, than to have left their selection to other City officials.

The writer believes the *Transactions* should not be burdened with a reply to the personalities contained in the discussion by Mr. O'Shaughnessy.

Mr. Rhodin.

CARL J. RHODIN,* M. AM. SOC. C. E. (by letter).—Broadly speaking, Mr. Cory's paper shows that increased water supply for the San Francisco Metropolitan District can be considered from the following sources:

1.—Further development of Spring Valley system;
2.—A supply pumped from the San Joaquin or Sacramento Rivers;
3.—Hetch Hetchy project;
4.—Other Sierra or Mountain sources.

Each of these groups has its peculiar characteristics; they are here briefly summarized.

1.—Spring Valley System.—The storage system and sources of supply of the Spring Valley Company, without doubt, should be incorporated and form an important part of any increased water supply system for San Francisco. As it stands at present, the Spring Valley system can be readily extended to provide much additional water at relatively small cost, and not only can immediate needs be met, but demands for many years can be supplied by the purchase of this property.

Due to peculiar conditions in California, however, it is desirable to secure an additional source at this time for use when the economical development of the Spring Valley resources has been reached. This is due to the fact that large quantities of water in the streams of California are continually being appropriated for permanent use in irrigation. When once applied in this way, the water cannot be taken for other purposes without great economic loss to agriculture and the

* San Francisco, Cal.

payment of immensely high condemnation prices by the parties desiring the water. Mr. Rhodin.

Furthermore, water for domestic purposes should not be taken from a territory where it can economically and readily be put to irrigation use, because the development of California depends so largely on the increased irrigation of its arid lands. Interference with this development by San Francisco indirectly hampers its own growth.

2.—Supply from the San Joaquin or Sacramento Rivers.—Popular opinion in San Francisco, generally, is not favorable to a supply from the lower reaches of the rivers. This feeling may not be warranted by the facts, but it exists, and must be considered. Popular support would be more readily found for a supply of mountain water than for a pump supply however pure it may be. Nevertheless, such a pumped supply, adequate in quantity and quality, does exist.

3.—Hetch Hetchy Project.—Under the Raker Bill, Congress has granted San Francisco the right to develop the Hetch Hetchy project, subject to certain restrictions, the most serious one being that the water requirements of 300 000 acres of irrigation land must first be taken care of. Opinion is somewhat divided as to how much water is required for the irrigation districts. Estimates vary from 600 000 to 1 050 000 acre-ft. per year delivered to the land. The proposed San Francisco supply contemplates 400 000 000 gal. daily, corresponding to about 450 000 acre-ft. per year, making a total demand of from 1 000 000 to 1 500 000 acre-ft. annually. This does not include losses through evaporation, percolation, seepage, and leakage.

The water* discharged by the Tuolumne River, at La Grange, is shown in Table 8.

TABLE 8.

Water season.	Total run-off, in acre-feet.	Water season.	Total run-off, in acre-feet.	Water season.	Total run-off, in acre-feet.
1895-96	1 590 000	1900-01	2 720 000	1905-06	3 530 000
1896-97	2 440 000	1901-02	1 610 000	1906-07	3 760 000
1897-98	960 000	1902-03	1 970 000	1907-08	1 070 000
1898-99	1 330 000	1903-04	2 660 000	1908-09	2 650 000
1899-1900	1 630 000	1904-05	1 720 000	1909-10	2 080 000
				1910-11	3 410 000

From Table 8 it is apparent that, to supply the total requirements, a very complete development of the water-shed will be necessary.

Under the Act, the burden of providing funds for construction rests on San Francisco, and it is apparent that although that city must carry the project to completion and be responsible for its success and all the contingencies, it has first claim to about the last third of the output only, the first two-thirds going to the irrigation districts.

* Water Supply Paper No. 299, U. S. Geological Survey.

Mr. Rhodin.

Under these conditions, even if development costs were low, the nature of the restrictions are undoubtedly serious. As the development costs are far from being relatively low, the advisability of proceeding slowly with the project becomes very important. If it is found that Hetch Hetchy is the best available source for an ultimate large supply, San Francisco must take the risk, but it certainly is advisable to investigate most carefully whether the risks, responsibilities, and expenses attached to Hetch Hetchy cannot be avoided.

4.—Other Sierra or Mountain Sources.—From the view-point that a desirable project should not deprive irrigable areas of future development possibilities, the water-sheds of the San Joaquin River and its tributaries do not seem to warrant further investigations. It is probable that the irrigable area in the San Joaquin Valley can ultimately consume all the economically developed and regulated flow available from these water-sheds.

The Sacramento Valley probably offers better possibilities, the McCloud River, Indian Valley, Yuba River, and other projects have been investigated more or less completely, and, in the main, offer possible solutions; but, some objection attaches to them, as there may be a partial demand on the water for irrigation use. Closer study may well prove some one of these projects acceptable, and free from the objections pertaining to Hetch Hetchy.

Another project, of which comparatively little is known, appears to possess features of considerable merit; this is the Eel River project, briefly mentioned by Mr. Cory. A portion of the drainage area of the Eel River was mentioned in the investigations preceding the passing of the Raker Bill, but did not receive extended consideration on account of insufficiency in size, and lack of cost data. A larger portion of the drainage area of the Eel River could be made tributary, and a much larger supply could be developed.

The Eel River rises in the Coast Range Mountains and flows northwestward through Mendocino and Humboldt Counties, discharging into Humboldt Bay. It passes through a forest country, where the future irrigation demands are not likely to become of importance. Construction offers no abnormally serious problems. From the diversion point, the conduit, in part paralleling the Northwestern Pacific Railway, would permit gravity flow for the greater portion of the distance to Point Pedro on the north shore of San Francisco Bay. The Bay could be crossed at this point by submerged lines to its eastern or Alameda County side. The conduit might then follow the shore line to Dumbarton, where the present Spring Valley lines into San Francisco Bay now cross; or a crossing could be made at a more northerly point. The total length of conduit would not differ materially from other proposed projects, and the profile is favorable, with no divides to cross.

Mr. Rhodin.

Closer investigations may reveal obstacles in the Eel River project, but the general features certainly seem to indicate sufficient merit to warrant a more serious attention than it has received.

Although the problem of an increased water supply to the San Francisco Bay cities has been "on the carpet" for many years, the discussions and investigations which have taken place have been so clouded by political and other extraneous arguments that it is quite possible that a number of meritorious projects have been entirely overlooked.

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INVESTIGATION OF THE PERFORMANCE OF A REACTION TURBINE

Discussion.*

By Messrs. R. C. Carpenter and R. L. Daugherty.†

R. C. Carpenter,‡ M. Am. Soc. C. E. (by letter).—The writer has been much interested in the discussion respecting the method of computing the turbine head which should serve as a basis of efficiency calculations. In Paragraph 6, the author points out the elements of the turbine head, Mr. Carpenter.

$$h = z_1 + p_1 + \frac{v_1^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

from which the efficiencies given in the paper were calculated. Mr. Taylor suggests that it would be fairer to the manufacturer if this were reduced by a correction for the velocity of discharge from the mouth of the draft-tube, in which case the head would be represented by

$$h = z_1 + p_1 + \frac{v_1^2}{2g} - \frac{v_3^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

In the writer's opinion, it is preferable to base the calculations for efficiency on the total head, as expressed by Equation (A), for although this gives an efficiency which is less than by the other method, it seems to be generally fairer than the method preferred by Mr. Taylor. It is true that the turbine builder may not be responsible for the discharge losses of the draft-tube, but it is also true that the draft-tube is an essential part of the turbine and is usually designed by the builder of the latter. The form of the draft-tube, as well as the construction of the turbine runner, affects this discharge loss, and for that reason

* Discussion on the paper by R. L. Daugherty, Esq., continued from February, 1915, *Proceedings*.

† Author's closure.

‡ Ithaca, N. Y.

Mr. Carpenter.

the designer should assume the responsibility for the efficiency of the draft-tube. From the standpoint of the purchaser, it is generally desirable that the efficiency which is stated in any proposal, should represent as nearly as possible the total efficiency of the operating plant. In the case of high heads, the effect on the efficiency of using either method of calculating the head is extremely slight.

Regarding the capacity of pumping engines, similar discussions have arisen, although in that particular case the points at issue related to methods of measuring the discharge water by piston displacement as compared with actual measurement. Naturally, the manufacturers have preferred the computations of the discharge by piston displacement, one result of which was to give slightly higher efficiency than the measurement by the actual volume discharged. On account of the difficulties involved in measurements, the piston-displacement method has been used regularly for capacity in the measurement of the pump discharge, and is usually so specified in contracts. The writer does not believe that any injury has resulted to the purchasers of pumping engines by contracting on the basis of piston displacement, and he feels that such would be the case if the head, as specified by Mr. Taylor, were used as the basis of calculation in the sale of turbines, provided such practice was general and was thoroughly understood by purchaser and manufacturer.

The characteristic curves (Fig. 12) which the author has worked out from the turbine test are extremely interesting; in the writer's opinion, they give valuable results which are applicable in the study of the performance of turbines generally, and are quite impossible to obtain with the data usually available in a turbine test.

Mr. Daugherty.

R. L. DAUGHERTY,* ESQ. (by letter).—The writer agrees with Professor Zowski that the draft-tube is an integral part of the turbine, and that all losses connected therewith should be charged to the turbine; but, in case it is desired to compare two turbines which have had to be equipped with radically different draft-tubes, due to differences in setting, it would be only fair to eliminate any differences due to the draft-tubes alone. In other words, we are then concerned only with the efficiency of the gates and runner, not with the complete unit. The question is, therefore, how far shall we proceed in the elimination of the draft-tube. This point will be considered subsequently.

It is generally admitted that, for ordinary purposes, the head charged against a turbine should include the draft-tube. The difference of opinion between Mr. Taylor and the writer is as to what constitutes this head. The writer contends that the head consumed by the turbine (including the draft-tube) is the total fall minus the

* Ithaca, N. Y.

penstock losses. The exit velocity at the mouth of the draft-tube is regarded as a loss chargeable to the turbine, owing to the failure of the turbine runner and the draft-tube to abstract all the kinetic energy from the water and reduce this velocity to zero. To be sure, it is impossible to reduce this velocity to zero, but it is equally true that an efficiency of 100% is a physical impossibility in any real machine. The ideal turbine would have to abstract all the energy from the water. Mr. Daugherty.

It is the writer's opinion that the turbine is responsible for all the energy delivered to it; Mr. Taylor believes that it should be held responsible for the difference between the energy input and the energy output. The latter term is represented by the kinetic energy of discharge. Mr. Taylor contends that the velocity of the water at the mouth of the draft-tube is influenced by conditions of turbine setting over which the designer has no control. This is true; but it is also true that this velocity for a given setting is also a function of the discharge velocity from the runner and the rate of diffusion provided for in the draft-tube, both being matters in the hands of the designer. It seems to the writer, as well as to Professor Zowski, that the designer must make allowances in his guaranties, where the conditions of setting are unfavorable.

In the comparison of two turbines with different lengths of draft-tubes, which Mr. Taylor quotes, the writer was not sufficiently explicit in his language. The comparison was intended to apply to two identical turbines differing only in their settings. If the turbine runners were likewise decidedly different, Mr. Taylor's criticism would be entirely just.

At present the writer is conducting some tests on a single turbine with five different draft-tubes. Four are of the same length, and differ only in the flare. At one extreme is a tube of uniform diameter, and at the other is one having a flare of 12° for each side, or forming a frustum of a cone the vertex angle of which would be 24 degrees. Thus, there will be quite a variation in the value of v_3. The experimental work is not yet completed, and the writer regrets that he is at present unable to give numerical results which could be used to illustrate these points, but the following observations seem pertinent.

For a given quantity of water supplied to the turbine under a given pressure, it would seem as if the power supplied should be the same, whatever draft-tube is used. It might then be presumed that the efficiency of the turbine with these various tubes would be directly proportional to the power developed. Such would be the results obtained if the equation in Paragraph 6 were used, that is:

$$h = z_1 + p_1 + \frac{v_1^2}{2g} \qquad (A)$$

Mr. Daugherty.

The method of computation proposed by Mr. Taylor would give the equation,

$$h = z_1 + p_1 + \frac{v_1^2}{2g} - \frac{v_3^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

According to Equation (B), the turbine would be charged with a smaller quantity of power when equipped with the straight draft-tube than when fitted with one having considerable flare. Suppose, for the sake of argument, that the power output of the turbine was the same with all the draft-tubes. As the power output is the same for the same quantity of water delivered to the turbine under the same pressure, it would seem as if the efficiency ought to be unchanged. If Equation (B) were used, however, it would indicate that the turbine efficiency is better with the straight tube than when using a draft-tube with some flare. Actually, the power output will not be the same with the four different draft-tubes, but will be greater with a moderate flare than with a straight tube. The true gain in efficiency due to the use of a flaring draft-tube, however, will not be given by using Equation (B). The full benefit of the diffusing draft-tube will be realized only when the computations are based on Equation (A).

If different power outputs are obtained from this same turbine when fitted with various draft-tubes, all other factors being the same, then, by the use of Equation (A), we should obtain different values of the efficiency. These differences are due to the draft-tubes, but the efficiency of the runner alone is absolutely unchanged. Any method of computing turbine efficiency solely for the purpose of comparing two turbine runners, independent of the limitations imposed on them by their settings, should be such as to give identical values when applied to the case in hand. It is perfectly clear that, if the efficiency based on Equation (A) increases as the flare of the draft-tube increases, the use of Equation (B) will tend to reduce these differences. The writer, however, contends that the only true method is to eliminate the draft-tube and use the equation given in Paragraph 5, for the sake of comparison only. Whether or not this is really practicable is another question.

The writer agrees with Mr. Taylor that it is difficult to determine accurately the pressure at the top of the draft-tube, due to the unknown extent of rotation of the water at that point. Thus the head computed by the equation given in Paragraph 5 may be in error. Some recent computations on this, from another standpoint, have convinced the writer that the value cited on page 2481* is too low, as Mr. Taylor suggests. For this reason it may be, and doubtless is, better to use Equation (B), in comparing two turbines, than the equation given in Paragraph 5. This, however, is a case of sacrificing something which

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Daugherty.

is really logical in favor of something else approximately equal to it, which is more readily obtained. This is similar to the circumstances regarding the measurement of the discharge of displacement pumps, to which Professor Carpenter refers.

The objection which Mr. Taylor raises, in regard to the determination of the head at the top of the draft-tube, applies also to the determination of the velocity at its mouth. In the test of the Cornell turbine it was noticed, at zero speed on the one hand and at runaway speed on the other, that there was a very decided rotation of the water as it left the tube, the rotation being opposite in direction in the two cases. At the normal speed of the wheel, when the discharge from the runner is approximately "radial", this effect is a minimum, but whatever rotation exists at the top of the draft-tube will persist to the end, though slightly diminished in value. Thus the true velocity of discharge, v_3, will not be obtained by dividing the rate of discharge by the cross-sectional area of the mouth of the tube.

Mr. Taylor's observations of the cases where the draft-head is excessive are interesting. That defect in the setting would be covered by either Equation (A) or (B). The efficiency of the runner alone, however, is independent of this factor, and would suffer if Equation (B) were applied. Only the use of the equation in Paragraph 5, if the correct numerical value could be determined, would give the true efficiency of the runner, unaffected by the particular setting.

The writer wishes to express his appreciation of the value of Mr. Taylor's discussion, and especially of his interesting presentation of the effect of the specific speed of the turbine on draft-tube conditions.

The writer's conclusion is that Equation (A) should be used in the determination of the efficiency of a turbine in a given setting. He believes that he is upheld in this by practice and by Professor Zowski, Professor Carpenter, the Holyoke testing flume, and by Article 207 of the new test code of the American Society of Mechanical Engineers (1914). Where it is intended to compare two turbines which have widely different draft-tube conditions, Equation (B) may be used as being more practical than the other. It is true that, using Equation (B), the maximum efficiency of the Cornell turbine is 88.2%, as stated by Mr. Taylor.

As Professor Mead points out, several curves could be drawn through the three points determined for each gate-opening and the resulting characteristic curve is thus only approximate. The accuracy of the interpolated results will depend on the skill and experience of the investigator. A familiarity with the shapes of these curves will aid greatly in their proper construction. However, this paper was intended to point out the possibilities of this method of procedure and show how to obtain a complete set of curves with limited data, rather than to give an accurate analysis of this particular turbine.

Mr. Daugherty. The writer has indicated in the paper how a few other points can be obtained, and, with these additional data, there can be but little question about the exact nature of these curves. Only lack of time to make the test prevented the writer from obtaining a few such special points to illustrate the method further.

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PENSTOCK AND SURGE-TANK PROBLEMS

Discussion.*

By Minton M. Warren, Jun. Am. Soc. C. E.

Mr. Warren

Minton M. Warren,† Jun. Am. Soc. C. E. (by letter).—The writer is greatly indebted to those who have discussed his paper. The new points of view brought out are worthy of careful study by engineers interested in water-hammer.

The title of the paper and the many formulas quoted have led to some misunderstanding as to its object. The writer desired water-hammer to be the main theme, and considers his new Formula (*B*), $h = \frac{L\ V}{g\left(T - \frac{L}{a}\right)}$, to be equal in practical importance to the common pipe formula, $b = \frac{W\ Y\ D}{2\ f_s}$ (where f_s is the allowable stress in the steel, in pounds per square foot, the other letters being as given in the nomenclature). The résumé of other ordinary water-hammer formulas was given in order to point out the limitations which they all have.

The fact that the leading authorities on hydraulics have been using water-hammer formulas which vary by 100% shows the necessity of a thorough discussion of the theoretical and experimental results from all sources, and the desirability of establishing a correct formula, based on sound theory and backed by reliable experiments.

Joukovsky was the first to do this in the case of maximum water-hammer, and, with the discussions, it is hoped that this paper may be a step in the right direction in the case of the more usual slow closing of gates, or ordinary water-hammer.

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* Author's closure of the paper by Minton M. Warren, Jun. Am. Soc. C. E., continued from March, 1915, *Proceedings*.

† Boston, Mass.

Mr. Warren.

Dr. Webster's formula for surge tanks was developed with the hope that its derivation could be followed by the practical engineer. The writer has never been able to follow the mathematics used in deriving the other surge-tank formulas, and always hesitates to use a formula the derivation of which is not clear. If its derivation is unknown, the formula may contain assumptions which would make it dangerous to use in certain practical cases. For example, the formulas for ordinary water-hammer are constantly being misapplied by engineers ignorant of their basic assumptions, and the results of such work are often misleading and dangerous.

Mr. Johnson's Formula (49) is simpler to solve and gives results equal to those of Dr. Webster's formula, and, to those who can follow its derivation, it is undoubtedly preferable.

At the time the paper was published, neither the writer nor Dr. Webster had seen Professor Prasil's formula for surge tanks, developed along the same lines, but with such thoroughness and wealth of calculus as to be unintelligible to the writer.

Taking up the various discussions in order, there are certain points and questions which the writer will discuss briefly.

Referring to Mr. Church's discussion: Equation (G) is referred to as "the most nearly accurate" for two reasons: First, Dr. Webster derived it with fewer assumptions than there are in the other formulas given in the paper; second, its numerical results are nearer the truth than those of any of the other formulas, with the exception of Mr. Johnson's Formula (49). It is derived by assuming that the friction varies directly as the velocity of the water in the pipe, which will give results too large and, therefore, on the safe side, except for possible effects of governor action. Obviously, the other formulas, which give larger results, are less accurate. (See numerical examples on page 2551.*) Dr. Webster's formula refers only to the special case of turning on or shutting off the full load, and does not pretend to take account of such complications as governor action.

The writer wishes to thank Mr. Church for Formula (106), which seems to give results still nearer the truth. In form, it is identical with the writer's approximate Formula (50), but the constants are different.

In regard to water-hammer, Mr. Church's Formula (113) is the ordinary one given in the paper as (47), which is accurate for use when T is considerably larger than $\frac{2\,L}{a}$. It is discussed on page 2547.*

Referring to the discussion of water-hammer by Mr. Johnson, the writer was much interested to note that the calculations and experiments agree with his own, and that, in ordinary water-hammer, when

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Warren.

the gate is closed slowly, the pressure rises and then continues constant, with a flat top to the pressure curve. In a recent letter to the writer, Mr. Johnson says: "I have made measurements of pressure rise due to slow closing of gates many times in many different parts of the country * * * there is always a flat top of some extent on all the pressure curves, which I have observed." This fact is a very strong argument against Joukovsky's Formula (48) which is so strongly upheld in Mr. Vensano's discussion on page 697.*

As Mr. Johnson states, his Equation (6) is identical with Allievi's Equation (*C*), which is fully developed and discussed in the paper. For the most part, this formula checks the writer's Formula (*B*) very closely, but when it is used for values of T close to $\frac{2\,L}{a}$, it becomes incorrect and dangerous to use. This error is discussed on page 2538† and, as may be seen from the numerical examples on page 2551,† the formula violates the law of conservation of energy in certain cases, giving values greater than those obtained for instantaneous closing.

Mr. Johnson restricts the use of the formula to times equal to or greater than $\frac{L\,V}{1.5\,g\,Y}$. Even this does not do away with the possibility of error, as may be seen from the following numerical cases.

Take the data on page 2549,† a 10-ft. pipe, $\frac{1}{2}$ in. thick, 400 ft. long, with water flowing at 6 ft. per sec.; $D = 10$, $b = 0.04$, $L = 400$, and $V = 6$.

First, taking a static head of 5 ft., Mr. Johnson's limiting time becomes $T = \frac{L\,V}{1.5\,g\,Y} = \frac{400 \times 6}{1.5 \times 32.2 \times 5} = 10$; therefore, the formula is not applicable unless the time of closing is 10 sec. or more. In this case it would not be usable in ordinary hydraulic work where the gate closes in from 2 to 5 sec.

Now take this same case with a static head of 400 ft., and the limiting time becomes 0.125 sec. The example worked out for $T = 0.32$ sec., on page 2551,† therefore, is allowable. The result is $h = 312$ ft., yet we know that as the wave has only just returned from the gate $\left(\text{as } T = \frac{2\,L}{a}\right)$, the maximum water-hammer, or $h = 465$ ft., must apply to this case. Therefore, in this case, Allievi's formula is 34% low, even with the restriction applied by Mr. Johnson.

In general, it seems to the writer that a water-hammer formula which varies its results with the static head, Y, must be inherently wrong. The gate, in closing, has merely to overcome the kinetic energy

* *Proceedings*, Am. Soc. C. E., for March, 1915.
† *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Warren.

of the moving water (friction neglected) which depends on its velocity. Why a formula should give a rise of 630 ft. under 100-ft. head and, with the same data, half as much under a 400-ft. head, is hard to imagine, yet Formula (C) gives this, as is seen on page 2551.* Presumably, this error is the result of the inconsistency of Allievi's assumptions, pointed out on page 2538.*

To Mr. Trautwine the writer is indebted for a careful and complete analysis of the phenomenon of wave motion in instantaneous closing of gates. His Fig. 6, for slow closing, shows a flat top to the pressure curve the length of which depends on $\frac{2\,L}{a}$, not on T, as in the writer's formula.

In connection with Mr. Williams' discussion, the writer wishes to mention a point of special importance in water-distributing systems, which Joukovsky discovered and proved both by theory and experiment. When a small branch pipe leads off a larger pipe in which water-hammer takes place, the small pipe will be subject to double the pressure in the large pipe, provided it has a dead end. This is more or less the principle of the hydraulic ram, and is due to the fact that when the wave reaches the dead end of the small pipe, the velocity must remain constant (zero) at that point. The only means by which this is possible is by starting a reflected wave having a velocity and direction opposite the first wave, the pressure of which is consequently the same. Adding the two pressures, we get double the pressure in the large pipe. If the small pipe has a still smaller pipe connected with it, the pressure may be quadrupled, and so on.

This phenomenon should also be carefully considered by hydraulic engineers when they take readings in instantaneous water-hammer from a pressure gauge set on the end of a small pipe leading from a penstock or wheel-case.

Joukovsky goes farther, however, and makes a statement which does not seem reasonable to the writer, namely, that when the wave returns from the dead end into the larger pipe, it may cause dangerous pressures therein. In sending the wave up the small pipe, the pressure in the large pipe is somewhat relieved, as if by a safety valve or air cushion. When the wave returns from the dead end, this fall of pressure may be more than counteracted, but it does not seem as if dangerous pressures would result, especially if there is considerable difference in the pipe sizes.

In reply to Mr. Anderson's questions on page 435,† the value of K, 42 400 000 lb. per sq. ft., or 294 000 lb. per sq. in., was taken from Mead's "Water Power Engineering". It would be safer to use a minimum value for this if any data are available from which to choose

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for February, 1915.

it. $E = 4\,000\,000\,000$ lb. per. sq. ft., or 28 000 000 lb. per sq. in., was taken from Kent's "Handbook". A maximum value would be safer to use. Mr. Warren.

As far as water-hammer formulas are concerned, E applies only to circumferential stretching of the pipe; and expansion or other circumferential joints do not enter into it. The longitudinal joints ought not to affect it appreciably. It is worth noting that in the numerical example for maximum water-hammer, given on page 2549,* a variation of 10% in either E or K makes a difference of from 2 to 4.5% in the answer to the problem. In ordinary water-hammer problems, the difference would be less, and in most cases negligible.

As regards the closeness of Fig. 2 to actual pressure curves, it is true, of course, that actual graphs would contain many irregularities, but from most formulas, as well as experiments, this seems to be the general form of the curve for slow-closing gates. Minor harmonics undoubtedly occur, and may be caused by air pockets, governor action, joints, or imperfections in the pipe.

As to nomenclature, the Greek ϵ would have been preferable to e, but a has been used by Uhl and by Allievi for wave velocity.

Mr. Vensano points out on page 697,† that, in Formula (H), for economical penstock diameter, B, the cost of pipe per foot of diameter per foot of length is a function of D which is unknown, and that consequently the formula should be deduced by assuming the cost of the pipe to vary directly as D^2, thus obtaining his formula.

The writer pointed out this fact on page 2548,* and stated certain reasons for giving the former formula the preference.

For a first approximation, Mr. Vensano's formula might be preferable, but for final work it is not true that the penstock cost varies as D^2, owing to the commercial sizes of plates and the allowance of metal for water-hammer and rust, which are not functions of D^2.

As a matter of fact, it makes very little difference which formula is used; either one is probably as accurate as the assumptions warrant.

Referring to Mr. Vensano's excellent exposition of Joukovsky's Formula (48), the writer believes that the mathematics is correct and the theory worth the most careful consideration by engineers interested in water-hammer. The writer disagrees with the resulting formula for three reasons:

First.—One of the assumptions seems to be wrong theoretically.

Second.—Practically, the shape of the pressure curve does not agree with any experiments the writer has ever seen, and the results are much larger than those found in actual practice.

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* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for March, 1915.

Mr. Warren.

Third.—Allievi, Church, Johnson, Mead, and other authorities give formulas which yield results about half as great as those of Joukovsky.

The seeming error in assumption is as follows: Mr. Vensano assumes that when the wave arrives at the gate, it is perfectly reflected, and the pressure falls, not only to zero, but below it, just as a pendulum swings across zero and up an equal distance on the other side. The first wave is started by the gate closing a small increment. This wave, though started by a local disturbance, spreads across the pipe, returns from the reservoir, and finds the gate still partly open. Therefore, only the part of the wave that encounters the gate reflects from it, the rest passes on into the turbine or open air, and its reflection is seriously impaired.

The perfect reflection encountered at the reservoir is due to the fact that the pressure there must be constant (atmospheric), though the velocity may vary. Similarly, at a dead end or closed gate the total velocity must be constant (zero), though the pressure may change.

At a partly open gate, however, as the pressure rises (due to the wave), the water shoots out at a higher velocity and relieves it somewhat. Certainly, the velocity is not constant or independent of the returning wave at the part of the gate that is still open, and consequently perfect reflection cannot take place.

As the gate approaches nearer and nearer to its closed condition, this argument has less and less weight, but there is another point worthy of attention. All experiments tend to show that, although the positive waves in pipes act very much as they should, according to theory, the negative waves are not equally accommodating and, in fact, rarely fall much below zero, however great the previous rise may have been. This may be seen very clearly by a glance at the actual graphs obtained in Joukovsky's experiments and reproduced in Simin's article.

The writer feels, however, that some slight reflex wave action may be present, and may account for some of the experimental numerical results which are greater than would be indicated by Formulas (*B*) or (47), as well as for some of the irregularities and harmonics noted in experiments. In other words, these waves, although not nearly as important as Joukovsky's formula would indicate, may explain discrepancies between Formula (*B*) and actual experiments.

If Mr. Vensano's deductions could be backed by actual experimental graphs, he would have a strong case, but as the writer has never before heard of a case where, in slow gate closing, the pressure fluctuated violently from zero to h, and as Mr. Johnson, in many practical cases, has always observed a flat top to the pressure curve, it would seem that the theory on which Mr. Vensano's formula is based, must be incorrect.

Mr. Warren.

In conclusion, the writer wishes to emphasize again the need of a general formula for water-hammer which will be applicable to the whole range of the phenomenon and which is, theoretically as well as practically, correct. Formula (B), if backed by careful experiments, would fill this need, and the writer hopes that some engineer who has the opportunity will make such experiments and present the results to the Society.

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PAPERS AND DISCUSSIONS

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COMPUTING RUN-OFF FROM RAINFALL AND OTHER PHYSICAL DATA

Discussion.*

By Messrs. Joel D. Justin, A. M. Strong, E. F. Chandler, and C. E. Grunsky.

Mr. Justin.

Joel D. Justin,† Assoc. M. Am. Soc. C. E. (by letter).—The author has shown a keen appreciation of most of the factors which cause the relations between rainfall and run-off to vary from one water-shed to another. In the writer's opinion, however, he neglects to a large extent one of the most important factors. In his paper, "Derivation of Run-Off from Rainfall Data", the writer found that slope and mean annual temperature (in the Eastern United States at any rate) were the chief factors which cause these relations to vary from one water-shed to another. It would seem to be almost self-evident that, rainfall and other factors being the same, the steeper water-shed will have the greater run-off and smaller evaporation.

The difficulties of making predictions as to the monthly distribution of run-off are very properly pointed out, and the necessity of taking account of the monthly temperature is emphasized by the author.

The writer agrees that it is not necessary to be able to estimate closely what the run-off was for any particular month of a certain year. It is sufficient, for all practical purposes, to decide on a distribution of run-off throughout the year, which, for the existing conditions of rainfall, temperature, and ground-water, might have taken place. The writer has shown‡ this by using mass-curves. He has also shown that, even without considering monthly temperatures, mass-

* This discussion (of the paper by Adolph F. Meyer, M. Am. Soc. C. E., published in March, 1915, *Proceedings*, and presented at the meeting of April 21st, 1915), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Harrisburg, Pa.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

Mr. Justin.

curves could be constructed, using the predicted monthly run-off as ordinate increments, which would give proper values for the storage required.

The author over-emphasizes the importance of the long-term mean (pages 631 and 632*). It is not the long-term mean which is so much to be desired, but the long-term record. To the writer, the term, "mean utilizable flow", is misleading. The most important thing to determine is the recurrence and interval of recurrence of series of dry years. For studying such problems, the mass-curve is by far the best and most convenient method.

As generally understood by engineers, evaporation is the difference between rainfall and run-off, and, as such, is capable of definite measurement on any water-shed where rainfall and stream-flow gauges are established. The author subdivides this evaporation into "evaporation from land surfaces" (to be corrected when there is a material quantity of water surface on the water-shed) and "transpiration". The writer believes this subdivision of the "losses out of rainfall" to be both an unfortunate and unnecessary complication of the subject.

If transpiration, which the author defines as "being the vaporization of water from the breathing pores or stomata of leaf and other vegetable surfaces", were a quantity capable of measurement and determination on different water-sheds, much might be said in favor of differentiating between it and evaporation.

The transpiration of any particular water-shed is, at best, only a guess. No one has ever measured the transpiration of a water-shed. Even if the transpiration of all the different kinds of plants was determined with a fair degree of accuracy, it would be difficult to apply this information to the entire water-shed. The segregation of transpiration from evaporation from ground surfaces would not be simple. Thus, on page 586,* the author mentions the data on water requirements of crops, recorded in *Bulletins* of the U. S. Department of Agriculture, and states that "the great difficulty encountered in the above mentioned experiments was that of differentiating between the evaporation from the soil and transpiration".

The true condition of this subject of transpiration is well shown by the diversity of the opinions of authorities referred to by the author. On page 588,* he states "Some writers say deciduous trees use more water than grasses and grains, and others claim just the opposite." * * * "If plants transpired a quantity of water equal to from one-half to two times the evaporation from an equivalent surface of water, as claimed by some experimenters, a great many streams in the United States that have a very appreciable sustained flow would become intermittent."

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* *Proceedings*, Am. Soc. C. E., for March, 1915.

Mr. Justin.

The writer is skeptical of the assertion that "yields of hay, grain, etc., become a convenient index to the approximate relative quantities of transpiration to be expected on different water-sheds". As one farmer will obtain a greater yield of a certain crop under exactly similar conditions of soil and moisture, it would be necessary to take into account the character of the population. Perhaps, it might be possible to derive a formula for the transpiration on any given water-shed, in which the following factors, represented mathematically, would appear: Thriftiness of inhabitants (as a coefficient to be determined by judgment); tons of hay, bushels of corn, oats, rye, wheat, etc., density of population per square mile, percentage of farmers who are graduates of agricultural colleges, tons of fertilizer per acre, gallons of alcohol consumed per capita per year, etc., etc.

Considering all the facts so well pointed out by the author, it would seem to be almost impossible to measure the transpiration of any particular water-shed. He, however, attempts this very thing with his base curve of transpiration (Fig. 17). This curve closely approaches a straight line and makes the transpiration very nearly proportional to the increase in temperature above 40° Fahr. Although the author does not particularly emphasize the fact, it is clear that this curve is not based on observed data, but on the assumption, that "the law first stated by Van't Hoff and Arrhenius, that most chemical reactions and physiological processes double in activity for every rise in temperature of 10° cent." is also applicable to transpiration.

Having thus obtained the theoretical transpiration for the different months, the author then modifies these figures by the use of a coefficient based on the "character and density of vegetation and length of growing season, with reference to temperature and hours of sunshine".

In a somewhat similar manner, curves are presented for the evaporation from land surfaces for various temperatures, and the results thus obtained for particular water-sheds are modified by the use of coefficients chosen by judgment based on the character of the water-shed.

In Tables 7, 9, and 11, the author has columns headed "Actual evaporation in inches, loss from land area". Any one examining these tables would surely think that the figures represented the measured evaporation from the land surface of the water-shed, in inches. It is, in reality, nothing of the kind, but is merely the author's estimate of the number of inches, out of the actual difference between rainfall and run-off, which he believes should be credited to evaporation from the land surfaces.

Thus, it is demonstrated that the differentiation between evaporation from ground surfaces and transpiration is a matter which is still in the realm of surmise. Inasmuch as even the author believes that transpiration "must necessarily depend largely on the same factors as evapora-

Mr. Justin.

tion", and in view of the impracticability of differentiating, except within very wide limits, between evaporation from ground surfaces and transpiration, what practical purpose is to be served by the complicating of this already complicated subject by the injection of transpiration?

In Table 5 is given a comparison between the mean annual run-off as computed by the author's method and as actually observed. The figures presented, except in the case of the McCloud River, show a startlingly close agreement between the observed and computed values, the discrepancy ranging as it does from a few hundredths of an inch to 1 in. This close agreement becomes the more remarkable when we consider the variation in the accuracy of the data involved. On some water-sheds, as pointed out by the writer,* the number of rainfall stations does not exceed 1 per 1 000 sq. miles of water-shed. Observations have generally been recorded by the native on whose property the gauge is located. Sometimes these observations are made accurately and honestly; at other times they are not. Cases have come to the writer's attention where the observer has deliberately "faked" a whole month or more of rainfall records. Frequently, the observer removes the gauge to what he considers a more convenient location near his house and outbuildings, where the observations can be readily taken by his 12-year-old daughter. Generally speaking, enough money is not available to insure the efficient supervision of work of this class. The writer believes that engineers should not expect most rainfall records to be within 1 or 2 in. of the truth. In some cases the discrepancy is much greater. On some water-sheds where observations on rainfall have been started in anticipation of the installation of an important water-power or water-supply project, the supervision has been close, and the records are all that could be desired. In the past, even some of the records of run-off have not been any too accurate. The author's division of the difference between the recorded rainfall and run-off into evaporation from water surfaces, evaporation from land surfaces, and transpiration, is, as has been pointed out, pure surmise, within very wide limits.

When the author compiled Table 5, he had before him the recorded rainfall and run-off for the various water-sheds considered. He then manipulated his curves, his coefficient, and his judgment to derive the three quantities, evaporation from water surfaces, evaporation from ground surfaces, and transpiration, always bearing in mind the fact, subconsciously or otherwise, that the sum of the three must approximately equal the difference between run-off and rainfall. Nothing could be more simple.

The use of well-seasoned judgment in the prediction of run-off is most essential, but what assurance can the author give of making a

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

reasonable prediction as to the transpiration on a water-shed where the run-off is unknown, when his figures for transpiration on water-sheds where run-off and rainfall are known are based on surmise, within very wide limits? Mr. Justin.

The author believes that there is no direct relationship between rainfall and run-off. The writer believes that, in his paper on the "Derivation of Run-Off from Rainfall Data", he proved that, within the limiting accuracy of the data involved, there was just such a relationship between rainfall and run-off, in the Northeastern United States, at any rate. He expressed this relationship by the formula, $C = 0.934 S^{0.155} \frac{R^2}{T}$, in which,

C = the annual run-off, in inches, on the water-shed.
R = annual rainfall, in inches, on the water-shed.
S = Slope of water-shed = elevation of highest point, minus elevation of lowest point, divided by the square root of the area.
T = Mean annual temperature.

Inasmuch as slope determines topography, and topography largely determines the character of the vegetal covering, this formula takes into account all the important factors which the author found to cause variation in transpiration and ground-surface evaporation. This formula has the additional advantage of being based solely on observed data, and therefore, the writer believes, can be safely used throughout the territory to which it was applied.

If extended to the Middle West and Far West, it should, of course, be checked by additional observed data, and the constant and exponents modified, if necessary, in accordance therewith.

It must not be expected that on any given water-shed a certain annual rainfall will always have the same annual run-off. The discrepancy, as previously pointed out by the writer, can generally be accounted for by beginning the water year on a date at which the ground-water level is not the same as at the beginning of the other water years. If the annual temperature of the different years had been taken into account, as suggested by the author, the agreement would probably be still closer.

The writer is very strongly of the opinion that on any given water-shed, if the rainfall for two water years is the same, the run-off for the two years will also be practically the same, provided that ground-water and temperature conditions are similar.

A. M. Strong,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has read with much interest the full analysis of the factors governing run-off given in this paper. In too many cases in the past, efforts Mr. Strong.

* Los Angeles, Cal.

Mr. Strong.

to compute run-off from precipitation have been unsatisfactory by reason of taking effect for cause. In using any formula or method for determining the relation between the two, a thorough knowledge of the laws governing evaporation, transpiration, and underground flow must be combined with a knowledge of the physical data for the drainage area under consideration. This information, accompanied by good judgment in comparing the known measurements of run-off with the records of precipitation, will give results of considerable value, where the proper coefficients are used, as shown by the author's computations. In any method, judgment and experience will largely determine the accuracy of the results, and are of more importance than the method used. In any case, the value of the results from computed run-off from any given drainage area can only be judged by comparison with known records. Great care should be used in comparing different drainage areas, even if adjoining, and no matter how similar they may appear.

Run-off from a given drainage area may be defined as that portion of the precipitation which remains on or near the surface of the ground too short a time to be lost or taken up by evaporation. A varying proportion of this is underground flow, and the remainder is surface flow; but that which is underground flow in one portion of the drainage area may become surface flow in another part. As the slow movement of underground water tends to equalize the seasonal variations in precipitation, the underground flow is fairly constant for any given drainage area, and largely governs the low-water flow of any stream. Flood flow is caused by surface run-off, and is governed by the irregular form in which the precipitation falls on the drainage area.

Evaporation and transpiration are very closely inter-related. Except in extreme cases, the character and extent of the vegetation on any drainage area are determined by the average quantity of water available for its growth. To a considerable extent, the variation in precipitation from year to year is met by variations in the extent and luxuriance of the vegetation, affecting both transpiration and evaporation. The ideal condition would be a continuous and even distribution of the precipitation during the growing season. Under these conditions, the character and extent of the vegetation would be such that it would take up all the water not lost by evaporation or carried off by deep underground flow.

Such an ideal condition is illustrated in the cienagas of the Southwest. These cienagas (springs or seepages) support a luxuriant vegetation consisting of a large variety of plant life. Usually, near the center, there is a little standing water, with its attendant plant growth, and, surrounding this, there are plants varying in kind and

luxuriance with the quantity of water available for them. Where these cienagas occur on a side-hill, as is often the case, the extent of the vegetation is such that transpiration and evaporation take up all the flow. It is often possible to drain these cienagas by tunnels or wells, and, in cases which have come under the writer's observation, the quantity of water so obtained shows that the transpiration and evaporation over the entire area affected by the flow must have had an average of more than 100 in. per annum. Mr. Strong.

If flood-water run-off is considered to be caused by the irregular distribution of the precipitation, it is evident that the conditions governing the run-off from any single storm are so variable that any exact computation is impossible. Annual run-off, being to a large extent the total of the run-off from a number of storms, gives a more average condition. Any attempt to analyze this average condition must take into consideration all factors, climatic and physical, and then, at best, is only accurate in as far as a short-time average approaches the long-time average.

E. F. Chandler,* Assoc. M. Am. Soc. C. E. (by letter).—The author should receive much commendation for the careful and painstaking work which he has done in developing and presenting clearly this method of computing probable run-off. His method is based on a well-arranged and correct chain of reasoning, and therein differs from the methods customarily used in the past by many engineers—methods which the writer has seized every opportunity to criticize as misleading and incorrect. Mr. Chandler.

The run-off is frequently regarded as a percentage of the rainfall. In the Eastern States, where careful consideration of the rainfall-run-off relation was first undertaken, and where the rainfall is so great that the run-off is usually more than half of it, such treatment did not lead to enormous percentage errors; but, in regions of smaller rainfall, it often leads to absurd results, and, in any region, it is wrong in principle. Although for convenience the run-off may sometimes be spoken of as a percentage, it is properly not a percentage, but a residual, the remainder of the rainfall after evaporation has occurred.

The author applies to different portions of the evaporation two different terms, evaporation and transpiration. Both rainfall and run-off can be measured with fair accuracy, and the evaporation from water surfaces can be measured with reasonably close approximation; but there is no practicable method for determining the true evaporation and transpiration over the ordinary land surface, except the indirect means of subtracting the run-off from the rainfall. Hence, the lateness of development of a well-founded method for making estimates

* University, N. Dak.

Mr. Chandler.

of evaporation and transpiration without the direct use of run-off records.

The writer endorses heartily nearly every step in the author's method and (with possibly a few minor exceptions) every statement in his paper, but is led to further comments in emphasis of some points and in warning against difficulties which must be guarded against in the application. These came to his attention during an attempt to apply this method for exemplification and verification in several drainage areas, with the physical characteristics of which he is thoroughly familiar and in which he has supervised the maintenance of continuous run-off records for 10 years or more. These streams were the Red River of the North at Grand Forks, N. Dak., having a length of record of 33 years, and a drainage area of 25 000 sq. miles; the Red Lake River, at Crookston, Minn., 13 years' record, 5 800 sq. miles; the Mouse River, at Minot, N. Dak., 12 years' record, 8 400 sq. miles; and some smaller streams. The writer has not had time to complete these tabulations fully, but the points to which attention was brought especially by these comparisons were as follows:

1.—That this method is quite laborious as compared with the conventional and incorrect method of merely applying a percentage coefficient to the total annual rainfall; but the results are so good as to be well worth the trouble.

2.—That much caution is needful in determining the rainfall figures on which to base the computations for a very large drainage area. Probably the only safe plan in studying such an area is to break it up into several smaller ones and compute each separately, for the normal rainfall differs so greatly in different portions of a large area that no average figures can logically be used. For example, the 25 000 sq. miles of the Red River area, has a normal annual rainfall of 25 in. at the eastern and 16 in. at the western margin; as a result, the total annual run-off is ordinarily between 6 and 3 in. at the eastern edge, and between 1.0 and 0.2 in. at the western edge. No system of averaging rainfall figures from the different sections of the drainage can be properly used for deducing for this entire region a single series of rainfall figures appropriate for this purpose; the differences between different sections of the area are comparatively too great.

3.—That if large lakes are in the drainage, lake storage must be taken into account, and this requires definite records of variations in lake level. For axample, the Red Lake River, with a drainage area of 5 800 sq. miles, has a mean flow of about 1 600 sec-ft.; one-third of its area is above the outlet of Red Lake, and the area of the lake is 440 sq. miles. It is reported that, in the course of a few years, variations of 2 ft. occur in the lake level; this is a storage equivalent to one-third of the total normal annual flow of the entire drainage area of the river

Mr. Chandler.

for a period of more than 16 months. The lake is in a timber country, mostly unsettled, or occupied only by scattered Indian settlers, and unreported changes in the discharge of the lake are likely to be caused by the temporary blocking of its outlet by sand-bars or drifting logs and brush, which, after a few months, would be washed away or passed around by the stream; if no lake records were kept, there might be unpredictable changes in the visible run-off farther down the stream, caused immediately by the lake, the ultimate relation of which to precipitation and temperature would be undiscoverable.

4.—The writer has often suggested that (in default of any more systematic and accurate basis of estimate), for each region, some figure might be determined as the measure of the annual rainfall usually needed to make good the evaporation and transpiration losses, and that any excess of rainfall above this figure may be expected to appear in the run-off. For Minnesota and the Dakotas, this figure would be 20 in. per year, or thereabouts. This assumption is more nearly correct than to estimate the run-off as a fixed percentage of the rainfall, but it must not be assumed to be an accurate statement or anything better than a mere makeshift basis for rough estimates; topography, seasonal distribution of rainfall, rate and number of heavy rainfalls, and temperature, ought to be considered, as the author has done.

It is unquestionable, however, that in the extreme Northern States, if the annual precipitation is much less than 20 in., the run-off becomes almost inappreciable if the rains are well-distributed and not violent and do not occur when the ground is frozen. In other words, after evaporation and transpiration losses, the remainder reaching the ground-water level is there so very small that scarcely any ground-water flows into the streams. For example, the Mouse River, at Minot, N. Dak., drains 8 400 sq. miles of country between latitudes 48° and 50°, the average rainfall for which is about 15 in. As remarked by Mr. Meyer, practically all the water in the usual stream in this latitude and region, between November 1st and March 1st, is seepage flow; in this case, the seepage flow, except after a season of unusually heavy rainfall, is so small that in some winters the river almost disappears, often having for months a flow much less than 5 sec-ft.; the long-period average (more than 10 years), for the 6 months from October to March, inclusive, is only 26 sec-ft., which is less than 0.03 in. of total run-off in the 6 months.

5.—That in the use, for any stream, of the method elaborated by the author, it is necessary to assume a coefficient by which the evaporations, as taken from the curve, are multiplied. This assumed coefficient is verified or revised by comparison of the figures secured by its use with those of actual measured run-off of the same stream,

Mr. Chandler

or a similar one in a near-by region. Unless such records of actual measurements of run-off are available, there is no check on the correctness of the assumed coefficients. In a region, such as the Northwestern Central States, where evaporation and transpiration take up nearly all the precipitation, an error as small as 5% in the assumed coefficient might double or triple the computed run-off, or might wipe it out entirely. Hence, it is evident that great care and caution must be exercised in using even this most logical and excellent method. In such a region of small run-off, unless actual records are available on the same or closely adjoining streams, so that the adopted coefficient can be verified by extensive comparison, no computations by its use ought to be permitted, for the results would be estimates so very rough that some method apparently much less refined, or even mere guesswork, would be likely to give as dependable results. Some actual run-off records are absolutely necessary as a starting point for the application of this new method in extending estimates into periods during which only precipitation and temperature records are available; otherwise, the computer is almost helpless.

This last comment is not intended to convey any general objection to the new methods, but rather as an expression of the writer's feeling that the continuance of gauging station and run-off records, based directly on actual measurements of flow, will be found to be more necessary and also more profitable than formerly; their value will be found to be much increased if this new method by which such advantageous extensions of run-off records can be deduced, comes into general use.

Mr. Grunsky.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—Referring broadly to practically any portion of the United States, it can be stated that wherever the annual rainfall exceeds 40 in., the portion thereof which will be lost to the stream by evaporation, transpiration, etc., is fairly uniform. The soil takes and gives to the air and to the plants a quantity of water which increases up to a certain limit with increasing quantities of rain. When the annual rainfall has reached from 40 to 50 in., however, there will be but little probability that larger quantities will cause any material increase in the evaporation from a soil already subject to frequent saturation, or that more rain will greatly increase the transpiration losses. If the rain is greater in quantity, there will be more rainy days and greater mean annual humidity—causes tending to prevent any considerable increase in the quantity of water which evaporation will keep out of the stream.

In other words, there is an astonishing uniformity (average values alone being considered) in the quantity of water which will be lost to the stream whenever the rainfall approximates or exceeds 40 in.

* San Francisco, Cal.

Mr. Grunsky.

per annum. This fact was pointed out by the writer long ago, and has guided him in formulating the following rule for approximating from the seasonal (12 months) rainfall the probable run-off:*

"The percentage of the seasonal (12 months) rainfall which appears in the stream as run-off, when the rain is less than 50 in., is equal to the number of inches of rain. When the seasonal rain is in excess of 50 in., 25 in. thereof goes to the ground (evaporates, etc.), the remainder is the run-off."

According to this rule, the run-off for 10 in. of rain in 12 months would be estimated at 10%, or 1.0 in.; for 30 in. of rain, it would be 30%, or 9 in.; for 60 in. of rain, it would be 60 — 25 = 35 in.

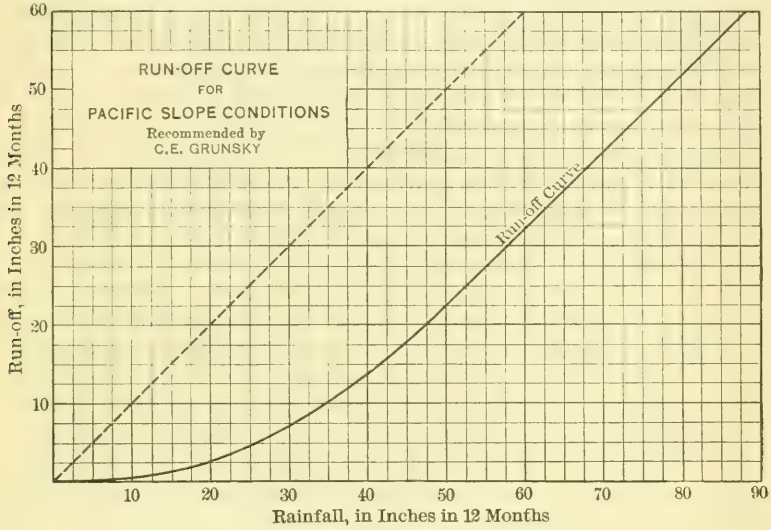

FIG. 37.

This rule will give fairly close results for practically any portion of the United States, but, in using it or any run-off curve, it must be remembered that probabilities only are to be determined, and that the smaller the seasonal (12 months) rainfall the greater may be the percentage departure of the actual from the estimated run-off in any single period of 12 months.

Revised studies made by the writer of available run-off data in California, where the rains which produce the seasonal run-off fall between October 1st and May 1st, have led him to adopt the curve shown by Fig. 37, which departs somewhat from the curve based on the foregoing rule, as best expressing the relation (under the Pacific Slope climatic conditions) of probable run-off to the seasonal rain.

* *Transactions*, Am. Soc. C. E., Vol. LXI, p. 514.

Mr. Grunsky.

It will be noted that the total quantity credited by this curve to evaporation, transpiration, and other losses is 28 in. per year for quantities of rain in excess of 50 in. A curve for run-off under Eastern climatic conditions would probably show 25 in. of loss for rain exceeding 50 in. in 12 months. For a seasonal precipitation of less than 50 in., the curve here presented, or a slight modification thereof, will fulfill every ordinary estimate requirement where, from a known rainfall, predictions of the total run-off in 12 months are to be made.

The writer's experience indicates that it is usually difficult to determine the rain on the water-shed. Such curves as shown by him on Plate LIX, of Vol. LXI of *Transactions*, are by no means conjecture. They show the great effect of orographic features on rain distribution, and can be used to show the small dependence to be placed on the determination of rain from the recorded quantities at a few irregularly distributed stations.

The author is to be commended for the careful analysis which he has made, but the results noted in the paper should be used with caution until verified with better data than such at least as are given for the few California streams included in Table 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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NOMOGRAPHIC SOLUTIONS FOR FORMULAS OF VARIOUS TYPES

Discussion.*

By R. C. Strachan, M. Am. Soc. C. E.

Mr. Strachan.

R. C. Strachan,† M. Am. Soc. C. E. (by letter).—The discussion has brought out a number of interesting points, among which may be mentioned the demonstration by Mr. Nugent of the fact, suggested but not enlarged upon in the paper, that some formulas may be charted in more than one way. This is a distinct advantage, as it enables one to choose the form best adapted to his purposes.

Mr. Eliot has well shown how the use of natural scales may be extended to several formulas in addition to those similarly charted in the paper. In reference to this the writer would emphasize the fact that the nomograph is a generalization which includes the many forms of the slide-rule as particular cases. The slide-rule is essentially an instrument designed for adding logarithms graphically; and its parallel scales are in such relative positions that all isopleths—placed with the runner—are perpendicular to them.

The broader principle, however, enables one to use either logarithmic or natural scales, and to perform all arithmetical operations graphically. Scales may be laid down either on straight lines or curves, and they need not be parallel.

The result of any one of the many problems which are daily solved with the aid of the slide-rule, as well as the results of others not adapted to this means of solution, may be read with sufficient precision from a properly constructed chart.

* Author's closure of the paper by R. C. Strachan, M. Am. Soc. C. E., continued from April, 1915, *Proceedings*.

† Richmond Hill, N. Y.

Mr. Strachan.

Fig. 48, for the solution of an expression in the run-off formula proposed by C. B. Buerger, M. Am. Soc. C. E.,* illustrates the use of a curved scale. The isopleth as drawn gives the value of $q = 3.0$ when $N = 0.17$ and $P = 3.4$.

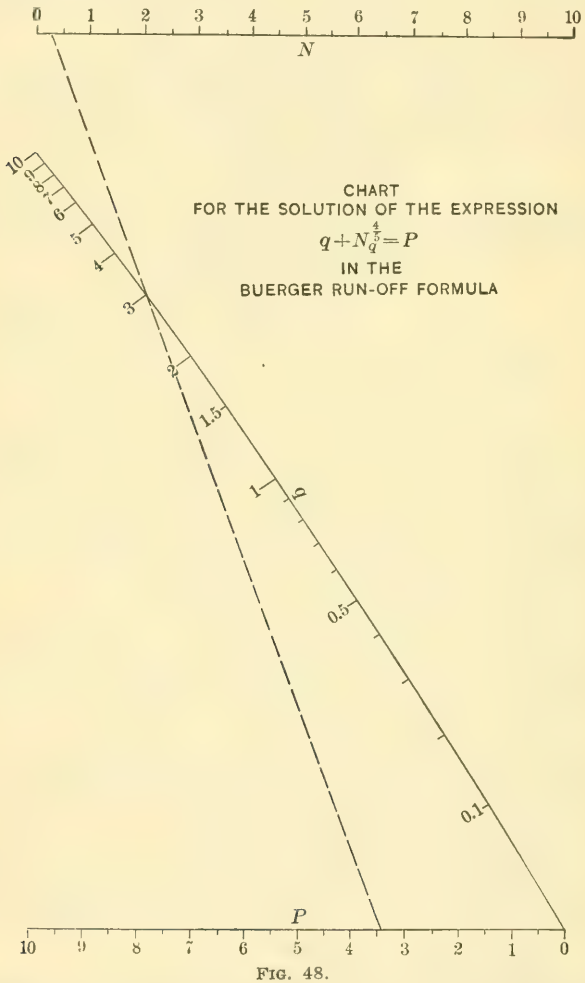

FIG. 48.

The method of obtaining the curve for the q scale applies to all expressions of the form:

$$x^n + Nx^m + P = 0$$

Mr. Hazen remarks that much of the ease of application of the nomograph is lost when more than three variables enter the equation.

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 1139.

This is true; but a more important consideration is the ease of obtaining solutions in a given case, with the nomograph as compared with other methods. Mr. Strachan.

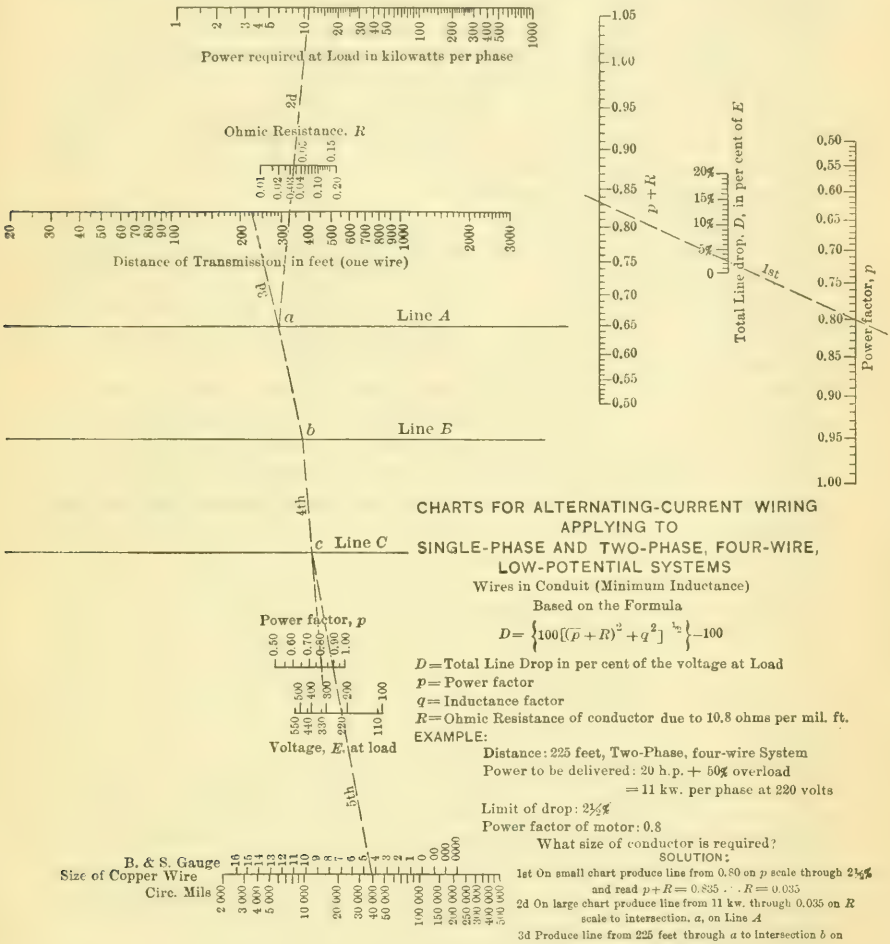
Power required at Load in kilowatts per phase
2d
Ohmic Resistance, R
Distance of Transmission, in feet (one wire)
3d
a
Line A
b
Line B
4th
c
Line C
Power factor, p
Voltage, E, at load
5th
B. & S. Gauge
Size of Copper Wire
Circ. Mils
p + R
Total Line drop, D, in per cent of E
Power factor, p
CHARTS FOR ALTERNATING-CURRENT WIRING
APPLYING TO
SINGLE-PHASE AND TWO-PHASE, FOUR-WIRE,
LOW-POTENTIAL SYSTEMS
Wires in Conduit (Minimum Inductance)
Based on the Formula
$D = \left\{100[(p+R)^2 + q^2]^{\frac{1}{2}}\right\} - 100$
D = Total Line Drop in per cent of the voltage at Load
p = Power factor
q = Inductance factor
R = Ohmic Resistance of conductor due to 10.8 ohms per mil. ft.
EXAMPLE:
Distance: 225 feet, Two-Phase, four-wire System
Power to be delivered: 20 h.p. + 50% overload
= 11 kw. per phase at 220 volts
Limit of drop: 2½%
Power factor of motor: 0.8
What size of conductor is required?
SOLUTION:
1st On small chart produce line from 0.80 on p scale through 2½%
and read p + R = 0.835 ∴ R = 0.035
2d On large chart produce line from 11 kw. through 0.035 on R
scale to intersection, a, on Line A
3d Produce line from 225 feet through a to intersection b on
Line B
4th On line through b on 0.80 on p scale mark intersection, c,
on Line C
5th Produce line from c through 220 volts and read 40 000 Circ.
Mils, or No. 4 wire required.
To find the line drop for a given wire, proceed in the reverse order

FIG. 49.

Fig. 49, designed by the writer, contains seven variables, and with it, by following the directions given, one may find the required size of wire conductor in an alternating-current system when the "line drop" is limited to a given percentage. The usual and accepted solu-

Mr. Strachan.

tion of this problem is by trial and error; which means that the better the guess the computer makes, the more rapidly will he obtain the answer. So far as the writer is informed, none of the many other diagrams invented for solving this problem does more than furnish a factor which must then be used in a numerical computation.

Mr. Eckersley objects to the words "nomograph" and "nomography" as misnomers, on the ground that the chart is not a "written law". Though this has nothing to do with the value of the charts, and though the writer is not in any way responsible for the name of the science, it is his opinion that the word "nomography" is the most appropriate one that could have been selected.

A mathematical law may be written in more than one way, as a statute law may be written in more than one language. With one object in view, the law is written as the graph or locus, and these words are perfectly well understood to refer to a certain method of representation. No exposition of the elementary principles of analytical geometry is required to make this clear.

When, with a distinctly different object in view, a new method of writing the law of the equation was invented, it was eminently proper that a distinctive yet descriptive word should be used in referring to it. The terms "graphical arithmetic", "graphic chart", and "metrical table" seem crude when compared with the word selected by the able originator of the method.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

FRANCIS HENRY HAMBLETON, M. Am. Soc. C. E.*

DIED MARCH 19TH, 1915.

Francis Henry Hambleton was born on December 28th, 1834. When he was 20 years old, he entered the Winans' Locomotive Shops, in Baltimore, Md., where the first locomotive used on the Baltimore and Ohio Railroad, as well as the famous "camel-back" engines, used on the same road for many years, were built. Mr. Hambleton remained in the shops for three years and was then transferred to the drafting-rooms, where he remained for one year. He was afterward sent to Russia by the firm, to assist in railroad construction, and remained in that country for several years.

Mr. Hambleton spent a year making a series of experiments on the friction and resistance of bodies passing through water. He also had charge, for the Winans, of the construction of the steamer *Winans*, in Baltimore, and of the erection of its machinery, together with the numerous experiments made with that vessel. Later, he was sent to England, where he had charge of the construction of another steamer, the *Ross Winans*, and of the working out of data accumulated during several years of experiments with that boat. In appearance, these boats were somewhat like the torpedoes used by modern submarines, and were known as "cigar ships". They were designed by the Ross Winans Company to lessen the time of crossing the Atlantic Ocean. While in England, Mr. Hambleton was also occupied in experiments in the mechanical puddling of iron.

On his return to the United States, he was engaged in the design of water-works, and, for a time, was employed as Resident Engineer on the Norfolk City Water-Works. He was also engaged in sanitary engineering work.

In 1877, Mr. Hambleton began his long career in the gas industry as Consulting Engineer for M. S. Frost and Son, of Baltimore, Md. This firm had a contract for laying about 50 miles of distributing mains for the Consumers Mutual Gas Light Company, and Mr. Hambleton designed and had charge of laying these mains, which were of large size for that day, the largest being 30, 24, and 20 in. in

* Memoir prepared by the Secretary from information furnished by George Beadenkopf, Chf. Engr., Consolidated Gas, Electric Light and Power Co., Baltimore, Md., and from papers on file at the Society House.

diameter, respectively. After the distribution system had been completed, he was put in charge of the construction of the manufacturing plant of the Consumers Company at Canton, from which station the first water gas was distributed to consumers in Baltimore, on January 28th, 1878. The Canton plant continued in operation until 1904 when it was closed as a manufacturing station.

In 1880, the Consumers Mutual Gas Light Company and the Gas Light Company of Baltimore were consolidated under the name of the Consolidated Gas, Electric Light and Power Company of Baltimore, and Mr. Hambleton was made Engineer of the new Company. In 1887 another company, the Chesapeake Gas Company, was added to the system, and Mr. Hambleton was retained as Engineer of the combined companies until the latter part of 1902, when he was appointed Consulting Engineer. He remained in this position until 1910 when he retired from active service. He retained his membership on the Board of Directors, however, until his death, having served as a Director of the Consolidated Company for 12 years.

During the last 38 years of his life Mr. Hambleton had been identified with the manufacture of gas in Baltimore, but, he also took an active interest in the construction of the cable railroad on the various car lines of that city when rapid transit was being introduced.

Mr. Hambleton enjoyed the reputation of being an Engineer of varied and unusual attainments, his experience having been very wide and covering many fields. He was held in high esteem by his fellow-directors and his associates, and enjoyed the confidence of all who knew him.

The following is quoted from the minutes adopted by the Executive Committee of the Board of Directors of the Consolidated Gas, Electric Light and Power Company of Baltimore, on the announcement of his death:

"For more than a quarter of a century the Company and the people of Baltimore benefited by the technical knowledge and engineering skill of Mr. Hambleton, who as Chief Engineer of The Consolidated Gas Company and later as the Consulting Engineer of this Company, brought to the solution of important engineering problems signal ability as an engineer; and whose breadth of view and public spirit characterized him as one of the best informed and most useful citizens of Baltimore. For many years a Director of this Company, the members of the Board, whose privilege it was to be associated with him, and the employees of the Company who knew Mr. Hambleton, will always retain and enjoy the enduring and inspiring impressions of his honesty of thought and methods of attainment."

Mr. Hambleton was elected a Member of the American Society of Civil Engineers on March 5th, 1873.

WILLIAM EDWIN MOORE, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1915.

William Edwin Moore was born in Pulaski, near Newcastle, Pa., on May 6th, 1863. He attended the public school at that place and afterward worked his way through the Academy at Newcastle and the Pennsylvania State College. He was graduated from the latter in 1889.

The first few years of his career were spent in general engineering work, and from 1896 to 1898, he was in charge of surveying parties on the Canadian Pacific Railroad. At about this time, he became interested in hydraulic work and specialized in that branch of engineering.

In 1898, Mr. Moore took charge of a large power and irrigation development for the Lewiston-Clarkston Company, at Clarkston, Wash. This was one of the early developments of its kind in the United States, and the many new engineering problems presented were met and handled by him in the most efficient manner.

The Lewiston-Clarkston work was not only difficult, but covered nearly the whole field of engineering. It consisted in the construction of an auxiliary steam turbine plant of 1 500 h.p.; two hydro-electric plants, the larger having a capacity of 2 500 h.p., under an effective head of 485 ft.; 14 miles of wood stave pipe, through one of the most precipitous and rugged canyons in the Northwest; a pumping plant; and 25 miles of smaller pipe; also the construction of two concrete dams, one for storage and one for a head-gate reservoir, together with an elaborate settling basin and forebay.

The construction of the large wooden pipe line, although not the largest in the country, presented as many problems as any other known line. This may be better understood when it is realized that 13 miles of the line crossed 56 deep gulches, which called for many bridges and inverted siphons, several of the latter requiring heavy steel pipe in place of the wood stave. This system of water-works not only furnished power, but irrigated several thousand acres of land covered by the same project.

In 1908, Mr. Moore opened offices as a Consulting Engineer, in Spokane, Wash., and, later, became a member of the firm of the Ingersoll and Bugbee Engineering Company, of that city, where he practiced until his death.

Mr. Moore was not only thorough in technique and practical in his design, but had good executive ability in controlling and holding the confidence of all his employees. Those in his employ have often remarked that, no matter how startling a problem was presented to

* Memoir prepared by M. L. Bugbee, Cons. Hydr. Engr., Spokane, Wash.

him, his cool, decisive, and quick reply would make one believe that he had always been familiar with the occurrence.

He was deeply interested in the Spokane Association of Members of the American Society of Civil Engineers, and up to the time of his death had done much to promote a Water Code for the State of Washington.

Mr. Moore was a most genial man and respected by all who knew him. He found his chief pleasure in his home. He was an exemplary character, the very soul of honor, and prosecuted his work in a most zealous manner and with untiring energy.

He was married, in 1898, to Miss Evelyn King, of Boston, Mass., who, with two sons, Allen King Moore and Donald Edwin Moore, survives him.

Mr. Moore was elected a Member of the American Society of Civil Engineers on February 7th, 1906.

ROBERT LELAND READ, M. Am. Soc. C. E.*

DIED JUNE 9TH, 1912.

Robert Leland Read was born in Manchester, N. H., on July 12th, 1841. His ancestors were distinguished in the early history of the country and, in consequence, he became a member of the Society of Colonial Wars and several other prominent organizations interested in the Colonial and Revolutionary periods. After his retirement from the active duties of his Profession, he was frequently present at their meetings in Boston, and was always actively sympathetic in their proceedings.

Mr. Read was educated at Dartmouth College, and was one of the early graduates of the Chandler Scientific School. In 1864, there were seven graduates, Mr. Read being the only one of that number who entered the Profession of Civil Engineering.

Soon after leaving his studies, in January, 1865, he was fortunate in securing a position as Assistant Engineer on some surveys connected with the Indianapolis, Cincinnati and Lafayette Railroad, and he remained with this road for several years. The Civil War was just ending, and the railroads were passing through a trying period, with little money available either for maintenance or construction; it is rather surprising, therefore, to find that the compensation given Mr. Read was $100 per month and all expenses paid. A short time afterward, we find, from his notes, that the regular pay for an instrument-man was $130 per month and expenses. The panic of 1873, however, caused these prices to drop suddenly and permanently.

* Memoir prepared by Desmond FitzGerald, Past-President, Am. Soc. C. E.

The early railroad experiences of Mr. Read follow closely the writer's experiences in Southern Indiana during the same period, and are so characteristic of local conditions of that time, that he has ventured to rehearse some of Mr. Read's notes and later to quote several lines in full.

A majority of the railroad officials prided themselves on being practical men, with no scientific training. They looked on an engineer as a necessary evil in construction, to be quickly discharged as soon as the roadbed was finished. The master mechanics were generally promoted from the ranks of locomotive engineers and, as a rule, had but a poor opinion of book learning. The master mechanic of Mr. Read's road once said to him: "I never bother myself with what other master mechanics are doing, except to make my engines different from theirs, so the president and superintendent of our road will think they have a smarter man than the other roads have."

Under this system, when the "Big Four" roads were consolidated, out of 500 locomotives, there were 115 different types. They were small wood burners, capable of drawing only from three to four cars.

Soon after Mr. Read began his railroad work, he was sent to make surveys of a branch line from Fairland to Martinsville. In regard to this branch, he wrote:

" * * * the first twelve miles being new line and the last twenty-six the rebuilding of an old road that had been originally laid with strap rail and abandoned many years previously when the track became so bad that a man had always to walk ahead of every train with spikes and maul to fasten down the loose rails.

"The Resident Engineer ran the transit and I the level. Just before the location was finished, the Resident Engineer went on a spree and I never saw him on the work afterward. The Chief Engineer, who only spent a few hours on the work during the whole period of construction, placed me in charge, but would give me no assistant, so I had to do all the instrument work, inspection, and office work; this meant at least sixteen hours a day. Our party was not allowed a horse, so we had to walk the thirty-eight miles between the ends of the work, living on the country, with all that that meant in Southern Indiana, which was principally settled by poor whites from Kentucky."

Following these early experiences, Mr. Read became Chief Engineer of several railroads connected with the "Big Four" system. From October, 1870, to May, 1871, he was engaged as Contractor on the construction of twenty-nine bridges on the Atchison, Topeka and Sante Fé, after which he was connected with the Ohio and Mississippi Railroad, and the construction of the Storrs Incline at Cincinnati. Later, he became Chief Engineer of the Indianapolis Belt Railway, and, in the latter capacity, was associated with General Harrison, the

Attorney for the enterprise, who, later, became President of the United States.

On September 2d, 1869, Mr. Read was married to Miss Abby H. Eastman, of York, Me., the daughter of Dr. Caleb and Adeline Talpry Eastman. She died on August 30th, 1893.

When Mr. Read retired, he removed to Malden, Mass. He was a prominent summer resident of Biddeford Pool, Me., where he was active in good works of every kind. In his will there was a bequest of $1 000 to Dartmouth College, "the income of which is to be used as an annual prize for the best work in descriptive geometry."

The foregoing are the hard and fast outlines of a busy life, but they poorly portray the real man, whose kindness and charm of manner endeared him to all with whom he came in contact. The latter part of his life, after the struggle and heat of the day had passed, must have been in many ways very lonely, but he was never heard to complain, and maintained a calm and cheerful spirit to the end. He died in Portland, Me., on June 9th, 1912.

Mr. Read was elected a Member of the American Society of Civil Engineers on September 2d, 1874, and served as a Director in 1892 and 1893. He was associated with other Engineering Societies, and served a term as President of the Engineers' Club of Cincinnati.

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